

JOURNAL *of the*
BOSTON SOCIETY
OF
CIVIL ENGINEERS



OCTOBER - 1947

VOLUME XXXIV

NUMBER 4

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715 Tremont Temple, Boston

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Entered as second-class matter, January 15, 1914, at the Post Office
at Boston, Mass., under Act of August 24, 1912

Published four times a year, January, April, July and October, by the Society
715 Tremont Temple, Boston, Massachusetts

Subscription Price \$4.00 a Year (4 Copies)
\$1.00 a Copy

Acceptance for mailing at special rate of postage provided for in
Section 1103, Act of October 3, 1917, authorized on July 16, 1918.

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THE HEFFERNAN PRESS

WORCESTER, MASS.

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS

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RECENT DEVELOPMENTS IN CONCRETE DURABILITY

BY HENRY L. KENNEDY*

(Presented at a joint meeting of the Designers' Section and the Transportation Section, B.S.C.E., held on December 11, 1946.)

CONCRETE durability, particularly as measured by freezing and thawing, has of late been receiving more than its fair share of attention in the literature, research laboratory, and in the field. This emphasis results from the recent advances in concrete technology in developing methods of assuring the resistance of concrete to freezing and thawing and chloride salt action. The methods thus far developed can be briefly described as the careful utilization of air entraining agents, either added at the concrete mixer or interground with portland cement to make air entraining cement. However, before discussing air entraining agents in detail, it may be well to summarize briefly the present status of additive materials in general.

For purposes of this paper, additive materials are classified as two general types: (1) admixtures, which are defined as materials added to concrete at the concrete mixer in order to change or modify certain specific properties of green or hardened concrete, and (2) addition agents, or simply additions, which are interground with portland cement at the cement mill for the same purpose as admixtures. Theoretically, all additions can be considered as admixtures, since their ultimate objectives are the same, but because of practical factors involved in the manufacture and storage of portland cement not all admixtures can be interground with cement, and hence cannot be classified as additions. It may be noted in passing that a third type

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of additive material exists, namely, grinding aids, which are added during the grinding of portland cement clinker in order to increase grinding mill efficiency, but which have little or no effect on the properties of the resulting concrete when added in the amounts recommended for grinding aid purposes.

Admixtures and additions can be classified functionally as follows:

1. *Plasticizers*. These are used in order to promote better plasticity or workability of green concrete, either as defined by the commonly used slump test or as indicated by actual placeability of the concrete at the job, which in turn involves ease of flow of the concrete around reinforcing steel and cohesiveness or lack of segregation. Most of the commonly used additive materials are plasticizers, but they may have one or more additional functions.

Plasticizers may be subdivided into three groups. The first group consists of water-reducing agents, such as organic dispersing agents, which act on the cement and promote plasticity by increasing the amount or changing the nature of the early hydration products of portland cement. The second group consists of air entraining agents, such as resins, fats, sulfonated hydrocarbons, wetting agents, which promote plasticity by introducing quantities of microscopic air bubbles or "ball bearings" in the concrete and whose action depends largely upon the presence of sand in the concrete. At this point, it may be well to note that some dispersing agents may act as wetting agents, and vice versa, depending upon the percentage of the additive material used.

The third group comprises those materials which are generally added to concrete in relatively large percentages or are used as replacements for portland cement and which promote plasticity by virtue of their fine state of subdivision or the spherical shape of the individual particles. Diatomaceous earth is an example.

2. *Accelerators and Catalysts*. These materials are used to enhance the strength, particularly early strength, of concrete. The first combines chemically with the hydration products of portland cement and loses its identity in the hardened concrete. Accelerators must be used in relatively large percentages (1-3% by weight of cement) in order to accomplish their objective. Calcium chloride, for example, has been used for many years.

Catalysts act to hasten the reaction between portland cement and water without being consumed in the process. They are characterized

by the relatively small percentages required to accomplish their objective. For example, one catalyst available today when used in amounts of 0.01% by weight of cement will cause sizeable increases in the early strength of concrete.

3. *Air Entraining Agents.* These have already been discussed in part under "Plasticizers." They are currently being used to improve durability of concrete, particularly resistance of concrete to alternate cycles of freezing and thawing and to the scaling action of chloride salts, in which connection improvements of the order of magnitude of 600-700% are obtainable. They also improve the resistance of concrete to sulfate waters and sea water, but the extent of this improvement has not yet been definitely established. It should be noted that all plasticizers are not air entraining agents.

4. *Water Repelling Agents.* Water repelling agents are generally interground during the manufacture of masonry or mortar cements. Their primary purpose is to impart a "fattiness" and cohesiveness to the mortar which is much desired by brick layers and masons. Such agents also cause water repellency, i.e., resistance of the cement to wetting by water during the mixing operation, but this property is incidental and not desired by many users of mortar cement. Water repellency is not necessarily proof of impermeability of the hardened mortar, although temporary resistance to the absorption of water under low pressures is frequently obtained. Stearates, fats and rosin are examples. Such agents are usually air entraining agents.

5. *Agents to Cause Controlled Expansion.* Aluminum reacts with the calcium hydroxide liberated during the hydration of portland cement, thereby causing hydrogen gas to be evolved (1). If small percentages of aluminum powder (.001% by weight of cement) are mixed with concrete, the evolved hydrogen gas is largely retained within the concrete and causes expansion of the concrete during initial setting. If the proper percentage of aluminum powder is used, the amount of expansion will compensate for the shrinkage normally resulting when concrete hardens.

Aluminum powder has been applied in this way with considerable success in improving the bond between concrete and reinforcing steel. It has been applied with less success in the grouting of machine beds, field block outs, etc., where it has been found that the evolved gas tends to escape from the thin layer of grouting material before

hardening and thus permits partial settlement. Hence, for this purpose, grouting materials consisting essentially of metal filings or equivalent, whose expansion in concrete does not depend upon gas evolution are preferred by most engineers or contractors.

6. *Pozzolan Materials.* The partial replacement of portland cement by a pozzolan material has long been recognized as a desirable procedure in order to reduce heat evolution and volume change with its attendant cracking which is frequently encountered during the hardening of mass concrete. Furthermore, the ability of pozzolans to combine with free lime and lime compounds liberated during the hydration of portland cement is also of great importance, for by this means some of the soluble, nondurable hydrated cement compounds are made insoluble and hence more resistant to the leaching action of corrosive waters.

The major objection to the use of pozzolans is the serious deficiency in early compressive strengths of concretes containing them, this deficiency causing difficulty in the removal of forms. Three-month or later compressive strengths of portland-pozzolan cement concretes may be the same or greater than those of corresponding plain concretes, despite low early strength, provided continuous curing of the concrete is applied during the three-month aging period. It should be noted that the slower rate of early hydration aids in reducing shrinkage and cracking in mass concrete. The deficiency in early strength can be compensated for by the use of CaCl_2 , but this will defeat the purpose of obtaining reduced shrinkage and cracking.

Pozzolan materials are sometimes used as additives to portland cement in relatively small percentages, but they are more generally used as replacements for portland cement in amounts of 20-30 per cent by weight of cement. Flyash (i.e., Cottrel precipitator dust), pumice, volcanic tuff, are examples, the first-named being at the same time a plasticizer because of the spherical shape of the particles.

7. *Natural Cements.* Natural cements are seldom used as additives to portland cement, but because of their functional similarity to pozzolans are included in this summarization. Reduced heat evolution and volume change of mass and highway pavement concretes are obtained by 20-30 per cent replacements of natural cement for portland cement.

It has been noted that natural cement, when used as a replacement for portland cement concrete, markedly improves the durability

of concrete (2). This may be attributable, in part at least, to the grinding aids customarily used in the manufacture of natural cement.

8. *Retarders.* The tri-calcium aluminate constituent of portland cement would cause immediate setting of portland cement upon gauging with water, if it were not for the retarding action of gypsum, which removes the tri-calcium aluminate from the reaction by combining with it to form calcium sulfoaluminate. Gypsum for this purpose is interground with portland cement in the amount of approximately 4% by weight of cement, in order to provide an SO_3 content in the cement of approximately 2%.

Oil well cements, which must have normal setting times when placed as pastes or slurries at the high temperatures existing at the bottom of 5000-16,000-ft. well casings, constitute a special case of the application of retarders as additions. For these exaggerated placing conditions, gypsum is not sufficiently effective and must be supplemented by gum arabic, borax or boric acid to obtain proper retardation of set. It may be of interest to note in passing that some manufacturers of oil well cement accomplish this purpose by changing the raw material proportions to eliminate tri-calcium aluminate in the finished cement or by grinding coarser, or both.

Combinations of the materials enumerated above may be used and are used to accomplish multifold modifications of concrete properties. In many cases, the classification of the agent constituting the maximum percentage controls the classification of the combination. In other cases, the classification of the combination is controlled by the constituent which has the predominating effect on the properties of the concrete. In any case, the classification is of minor importance as long as the overall effects of the addition or admixture are known and understood.

EARLY HISTORY OF DURABILITY DUE TO AIR ENTRAINMENT

In the late 1920's, Professor C. H. Scholer, of the Kansas State College, noted that a certain high early strength cement which contained an interground metallic stearate for the purpose of improving the warehousing characteristics of this high fineness cement appeared to produce concrete having measurably greater resistance to alternate cycles of freezing and thawing in plain water. He also observed the concretes showing improved durability were always lower in unit weight. However, since high density, i.e., high weight per cubic foot,

was at that time believed to be the criterion for durability of concrete, little or no publicity was given the results.

During the years 1932-1935, the above tests were duplicated by deliberately entraining air in concrete using various addition agents. In every case, resistance to freezing and thawing was improved, generally in proportion to the reduction in unit weight of the concrete. Since the Dewey and Almy Chemical Company had on the market at that time a combined dispersing and air entraining agent, the author was greatly interested in Scholer's findings and cooperated to a small extent in this further test work by testing dispersing agents combined with several wetting agents to cause various degrees of air entrainment in concrete. However, by 1935 it was obvious that the cement and concrete industry was not ready for an addition which caused such a radical departure from previous concepts of excellence in concrete, e.g., high density. Hence further investigation was practically eliminated and the combined dispersing and air entraining agent was replaced with a straight dispersive.

Meanwhile surface disintegration of concrete pavements, such as the Worcester turnpike, was becoming so serious that the use of concrete as a primary road material was threatened. It may be significant that surface disintegration or scaling became serious at about the time that cement plants started fine grinding of portland cement. To achieve the higher fineness required by the new specifications, many plants used coal as a grinding aid in order to avoid drastic reductions in grinding mill production rate. Coal or carbon inter-ground with cement inhibits the entrainment of air in concrete and therefore eliminates completely the small percentage of air normally entrained in plain concrete containing no additions. In view of the fact that later tests proved coal to be detrimental to resistance of concrete to freezing and thawing, it is conceivable that this factor was responsible for the sudden increase in concrete surface failures, for the inconsistencies attending such failures. Some explanation is necessary as to why the Worcester turnpike failed and the Providence turnpike did not.

During the middle 1930's, tests made by New York State indicated that concrete made from blends of portland cement and natural cement developed little or no scaling even when the large amounts of CaCl_2 were applied to the road surfaces to de-ice them in the winter. It was at first assumed that this improvement was due entirely to the

properties of the natural cement used but subsequent investigation revealed that it might also result from the action of the tallow which was interground with the natural cement to aid grinding and enhance plasticity. Even at this time entrained air was believed to be incidental and not the primary cause.

Coincidental with the above, was the observation by certain cement manufacturers that cements ground in old roller mills which leaked small quantities of lubricating oil into the cement being ground produced concretes of outstanding resistance to freezing and thawing. It was therefore deemed possible to improve portland cement *per se* by intergrinding a suitable addition agent at the plant. Research on this point was accelerated in order to develop an addition which would achieve the same results as tallow but which would be cheaper and easier to handle at the plant. Vinsol Resin was used for this purpose in much of the early work.

However, even at this time it was not generally accepted that entrained air was responsible for the improved concrete durabilities. Despite thousands of tests made with Vinsol Resin which showed consistent decreases in concrete unit weight checking previous results, improved durability was still attributed by many to some other but unknown action of the addition.

Having in mind the early results obtained in cooperation with Professor Scholer, laboratory investigation was continued by the Dewey and Almy Chemical Company and was directed specifically at establishing the function of our air entraining agents and the mechanism by which resistance to freezing and thawing and chloride salt action was improved. The next section of this paper discusses results obtained during this investigation and demonstrates clearly that entrained air and not some mystical action of the air entraining agent is responsible for improved concrete durability. From this it follows that any agent which will entrain the desired amount of air in concrete can be used satisfactorily provided that other desirable properties of concrete are not seriously impaired.

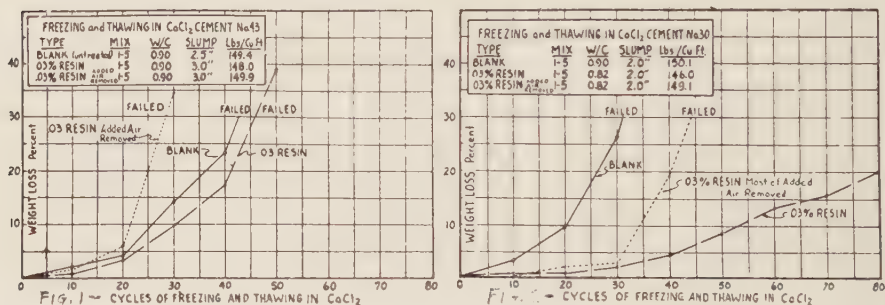
EFFECT OF AIR ENTRAINING AGENTS ON DURABILITY

If the improved durability of concrete in which is incorporated an air entraining agent is due to the agent itself and not the entrained air, it necessarily follows that concretes containing the optimum percentage of such an agent will be durable regardless of the amount

of air present in the concrete. Acting on this assumption, tests were made on plain concretes and concretes containing a resinous air entraining agent, as follows (3).

Comparable batches of concrete with and without an air entraining agent were made using $6\frac{1}{2}$ sacks of a given cement per cubic yard of concrete. A portion of each freshly mixed batch of air entrained concrete was subjected to vacuum in such a way that all or most of the entrained air but no water was removed, and then fabricated into specimens for freezing and thawing in a ten per cent solution of calcium chloride. Specimens were also made from the balance of each batch of air entrained concrete, from which the air had not been so removed. Control specimens, designated as "blank", of the same cement content but without the air entraining agent were made for comparison.

Typical data based upon comparative mixes of (1) identical water-cement ratio by volume, and (2) identical slump, are shown in Fig. 1 and 2. The concrete containing the added entrained air, as



indicated by the densities tabulated, show markedly improved resistance to chloride action, whereas the concrete containing the same air entraining agent but from which the air has been removed by evacuation shows little or no improvement. In fact, in Fig. 1, from which more than the added entrained air was removed, the concrete shows decreased durability.

The minimum percentage of air required to produce adequate improvement in the durability of concrete varies somewhat with different cements and different aggregates, but it appears to be approximately $2\frac{1}{2}\%$ by volume of the concrete. Fig. 3 presents in idealized form average data showing the relationship between per cent addi-

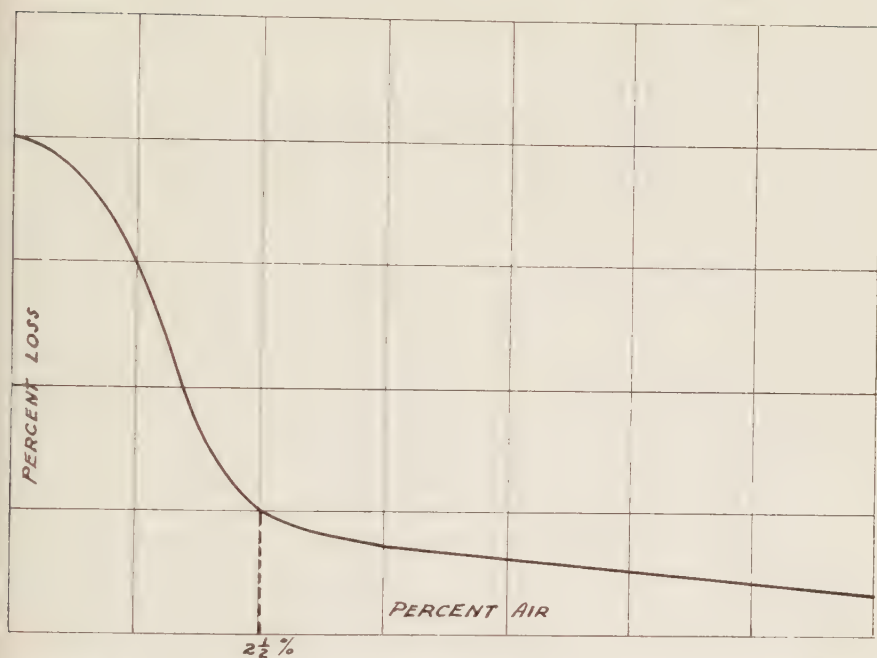


FIGURE 3— PERCENTAGE OF ENTRAINED AIR VS. CONCRETE DURABILITY

tional entrained air and per cent loss in dynamic modulus of elasticity or per cent loss in weight.

This figure shows that the entrainment of air in excess of $2\frac{1}{2}\%$ by volume of concrete does not materially improve durability, but the purposeful entrainment of less than $2\frac{1}{2}\%$ air markedly reduces concrete durability. This means that in order to provide a margin of safety to cover variations in methods of measuring entrained air, methods of placing the concrete, type and brand of cement, and type and gradation of aggregate, specification minimum limits should be set at 3% , a fact which is recognized by most specifiers of pavement and mass concrete today. Data have been obtained from time to time by various investigators which indicate that exceptionally good durability can be obtained with only 1% air, but in many cases the accuracy of the determinations of percentage entrained air may be open to question. In structural concrete, which is seldom subjected to the attack of calcium chloride and which may be subjected to relatively small amount of freezing and thawing in plain water or moist air, it

is possible that a minimum limit of 2 or $2\frac{1}{2}\%$ entrained air will provide adequate durability.

In view of the results obtained with evacuated concrete, and in view of the excellent correlation between percentage of entrained air and resistance to freezing and thawing, it is generally accepted today that entrained air and not air entraining agents per se are responsible for most of the improved durability. Therefore, if proper dispersion of the air through the concrete is obtained, entrained air will improve concrete durability regardless of the agent used to bring about the air entrainment, provided, of course, that the addition agent does not cause serious impairment of other necessary and desirable properties of concrete, such as strength, volume change, etc. These provisions are important because concrete may successfully withstand freezing and thawing and still disintegrate by other actions.

The effect of entrained air on the resistance of concrete to freezing and thawing and chloride salt action has been publicized to such an extent that further presentation of data in substantiation seems unnecessary. However, Fig. 4 is included because it shows the importance of permitting the concrete to dry out, at least partially,

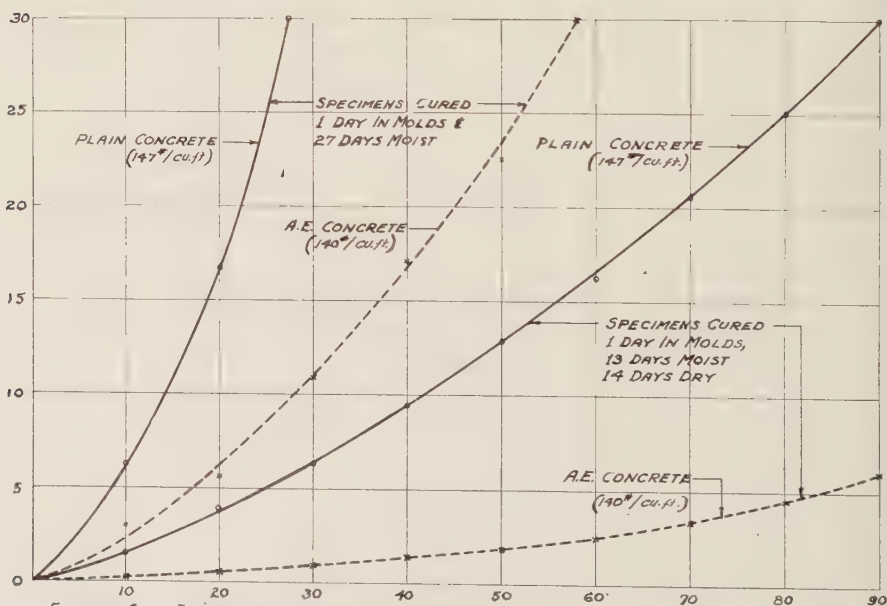


FIGURE 4 — DURABILITY OF PLAIN AND AIR ENTRAINED CONCRETE CURED UNDER VARIOUS CONDITIONS AND FROZEN AND THAWED IN 10% CaCl_2 SOLUTION

before submitting it to cycles of freezing and thawing, particularly when calcium chloride is involved at the same time. It will be noted that a marked improvement in the durability of the comparable concretes was obtained by dry-curing the concrete specimens for fourteen days prior to subjecting them to freezing and thawing in a 10% solution of calcium chloride regardless of whether or not an air entraining agent was used.

If hardened neat portland cement pastes containing entrained air are subjected to repeated cycles of freezing and thawing, little or no improvement in resistance to freezing and thawing is noted. In fact, results of such tests up to 105 cycles of freezing and thawing show that durability of the hardened cement paste is appreciably impaired, rather than improved, by the entrainment of air. This shows that cement paste must be diluted with fine and coarse aggregate in order to obtain durability benefits from entrained air.

EFFECT OF AIR ENTRAINING AGENT ON CONSISTENCY

In addition to enhancing concrete durability, entrained air also improves workability and placeability of fresh concrete. This is very important, not only from the point of view of eliminating honeycombing and segregation, but also because it makes possible substantial reductions in the water-cement ratio, which counteract in large measure the reduction in compressive and flexural strength normally encountered with entrained air. Hence an understanding of the mechanism involved in this improved workability is of great practical value in the utilization of air entrained concrete.

If neat portland paste is mixed in the presence of an air entraining agent, some air will be entrained with a reasonable amount of mixing, but it will be less in amount than in the same volume of concrete. This entrained air causes no increase in the plasticity of the paste but rather a decrease. Actually, the paste is stiffened just as when an equal volume of sand is mixed with the paste (3).

If, on the other hand, an air entraining agent is added to a stiff mixture of fine aggregate and water and given a reasonable amount of mixing, the mixture becomes very fluid, indicating that the action of the air entraining agent is different in sand-water mixtures as compared to cement paste. Figures 5 and 6 show how microscopic bubbles of air lubricate the fine aggregate by cushioning the contact between the grains of sand, thereby reducing particle interference to a mini-

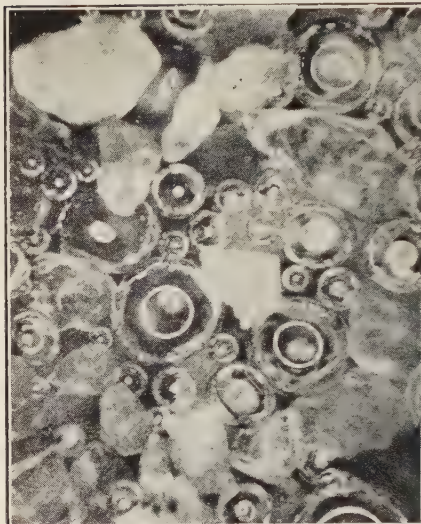


FIG. 5.—AIR ENTRAINED IN 30-50 MESH SAND PARTICLES.



FIG. 6.—AIR ENTRAINED IN 50-100 MESH SAND PARTICLES.

mum. It will be noted that in general the discreet air bubbles are considerably smaller in diameter than the sand particles, whereas the air bubbles evolved in neat cement paste are much larger in diameter. It may be noted that the sand grains in Fig. 5 are between 30 and 50 mesh sieves in size and in Fig. 6 are between 50 and 100 mesh in size.

The foregoing strongly indicates that the presence of fine aggregate is essential to the entrainment of air bubbles and that plasticity is improved by the lubricating action of the air bubbles on the fine aggregate. In this connection, it should be borne in mind that microscopic investigations reveal that little or no air is entrained in sand-water mixtures containing 30 mesh sand particles and larger. Hence the sand, and more specifically the 30-mesh to 100-mesh-range particles, is the predominating factor in the amount of air entrained with a given percentage of air entraining agent. Work of other investigators indicates that the effective range of sand sizes to produce maximum air entrainment may be 30 mesh to 70 mesh.

Further proof that air entraining agents work only on the fine aggregate and not on the cement is given by our simulated concrete tests. Figures 7 and 8 show simulated concretes made without any cement (4). The "concrete" in Fig. 7 consisted of sand, gravel, water,

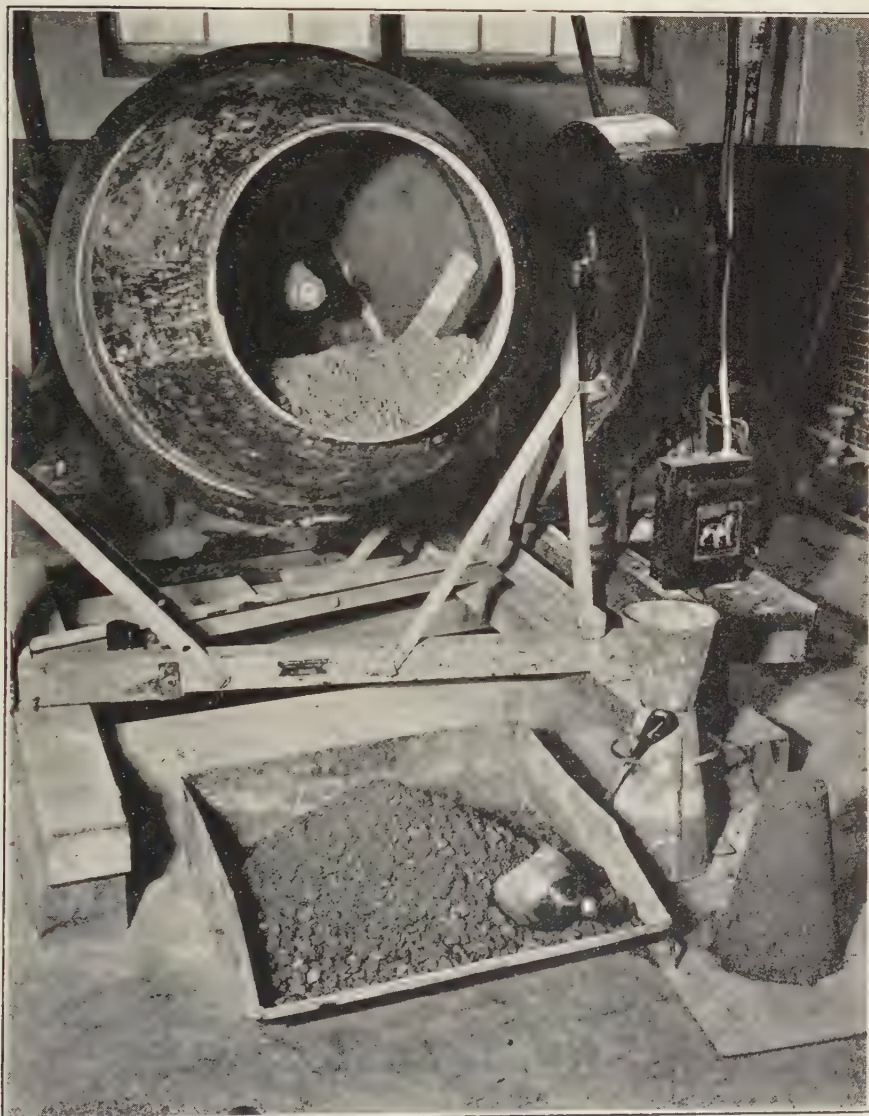


FIG. 7.—SIMULATED CONCRETE WITHOUT AN AIR ENTRAINING AGENT.



FIG. 8.—THE SAME SIMULATED CONCRETE WITH AN AIR ENTRAINING AGENT.

and a little carbon black (for coloring purposes) mixed to a very stiff, zero-slump consistency, and that illustrated in Fig. 8 is the same, except that an air entraining agent was added. The simulated concrete of Fig. 8 was actually selected by a contractor who was well-versed in concrete placing in the field as being far superior to a regular concrete made for comparison. In fairness to the contractor, it should be stated that he was unaware of the absence of cement in the simulated concrete.

Since entrained air stiffens the cement paste but plasticizes cement mortar, it should follow that lean concrete mixes will be plasticized to a greater degree than rich mixes with a given percentage of entrained air. Experience substantiates this. Consequently, less reduction in water-cement ratio can be made in concretes of high cement content than in concretes of low cement content, the percentage of entrained air being the same.

Since, for a given percentage of air entraining agent and identical mixing conditions, less air is entrained in cement paste than in cement mortar, it should follow that rich concretes will entrain less air per unit addition of a given air entraining agent than lean concretes. This also is borne out by experience. This condition is accentuated by the fact that the percentage of sand is normally less in rich concretes than in lean.

It has already been shown that air entraining agents work on the fine aggregate and therefore the amount of air entrained in concrete by a fixed percentage of air entraining agent is a function of, among other things, the percentage of sand by volume of the concrete. Furthermore, the grading or sieve analysis of the sand has a marked effect upon the amount of air entrained. This has already been implied by the discussion accompanying Figures 5 and 6 and is elaborated upon at this point because it probably has more effect on the amount of air entrained than any other single factor. Fig. 9 gives

SIEVE SIZE	Total Air - %	TOTAL AIR - %								Added Air - %, with	
	No Air Entraining Agent	With Air Entraining Agent								Air Entraining Agent	
		Method 1				Method 2				Ave.	
		Average	1	2	3	Ave.	1	2	3	Ave.	Ave.
No. 14 to No. 30	4.5	14.8	14.6	13.3	14.2	15.7	15.5	15.4	15.6	9.7	11.1
No. 30 to No. 50	5.9	20.0	21.0	22.5	21.1	21.6	22.2	21.0	21.6	15.2	15.7
No. 50 to No. 100	6.8	24.0	25.2	24.2	23.8	24.9	26.0	25.3	25.4	17.0	18.6
Passing No. 100	6.6	7.9	8.6	9.0	8.5	10.3	9.6	9.8	9.9	1.9	3.3

FIGURE 9 — Amount of Air Entrained by Different Sand Fractions in the Presence of an Air Entraining Agent.

in tabular form the relative percentages of air entrained in the various size fractions of sand by a given percentage of air entraining agent. These results were obtained during the early work in the Dewey and Almy Laboratories and have since been confirmed by other investigators (5).

It will be noted that very little air is entrained in the sand sizes below 100 mesh, indicating that the minus 100 mesh sand is not essential to the entrainment of air in concrete. Since the individual particles of entrained air have practically the same average diameter as the minus 100 mesh sand, and since they do the same job as the fine sand but do it better because of their flexibility and compressibility, it may well be asked if minus 100 mesh sand is necessary in air entrained concrete. The author believes it is not.

Minus 100 mesh sand is today required by many specifications in order to promote workability and reduce bleeding. However, 4% of entrained air improves workability and reduces bleeding to a far greater extent than the equivalent percentage of fine sand. In this day of washed concrete aggregates, the minimum specification limits of minus 100 mesh sand are sometimes difficult and costly to meet. Hence it is suggested that, although all investigators are not agreed upon the desirability of eliminating such minimum specification requirements for fine sand, construction costs can be appreciably reduced by utilizing entrained air to compensate for a deficiency in minus 100 mesh sand, especially in those locations where available deposits of sand are naturally deficient in fines.

The table also indicates that excessively coarse sands or excessively fine sands, as opposed to well-graded sands having normal percentages of the 30-50 and 50-100 mesh sizes, will decrease the effectiveness of air entraining agents, thus resulting in reduced air entrainment for a given percentage of air entraining agent. Such variations will cause corresponding variations not only in the workability of the green concrete but also in the durability of the hardened concrete, which is after all the primary objective of air entrainment. This clearly demonstrates that when bank run sands are used with a fixed amount of air entraining agent, large variations in percentage of entrained air, durability, and workability will result. In actual practice, percentage of air has been found to vary from 2.5 to 8% when using a given air entraining cement containing a fixed percentage of air entraining agent, due in large measure to variations in sand and richness of mix (6).

EFFECT OF AIR ENTRAINING AGENTS ON STRENGTH

When air is entrained in concrete, the concrete is said to "bulk", thereby giving a greater yield for given volumes of cement, aggregate and water. For example, quantities of materials which normally produce twenty-seven cubic feet of concrete in place, will instead yield $27 \div .04 \times 27 = 28.08$ cu. ft. of concrete if 4% of air is entrained. This means that for a given mix, the cement content and therefore the cement paste content of a cubic yard of concrete is correspondingly reduced. With this reduction in the unit volume of the cementing medium there must be a reduction in concrete strength, compressive, tensile and flexural, unless the strength of the cementing medium itself can be increased.

It has already been shown that entrained air increases workability to such an extent that appreciable reductions in water-cement ratio can be made and still maintain the desired workability. Also, the reductions in water-cement ratios are greater for lean mixes than for rich mixes. When such water cuts are taken, the strength of the cement paste and hence the concrete is increased, in the case of lean concretes almost enough to compensate for the reduction in cement paste content, and in the case of rich mixes only enough to compensate partially for the reduced cement paste content.

If in addition some of the sand or some of the coarse aggregate or some of both is eliminated from the mix, the cement content of the concrete can be returned to normal. The amount of aggregate eliminated must, of course, compensate volumewise for both the bulking effect of the air and the reduction in water content in order to restore to normal the cement content per cubic yard of concrete. By so doing, actual increases in the strength of lean concretes will be obtained and the strength of rich concretes not exceeding seven sacks per yard will be restored to normal and may even be slightly improved, all depending upon the amount of entrained air and the type of air entraining agent used. This statement, it will be noted, is qualified by imposing maximum limits upon the percentage of air permitted, these permissible maxima being greater for lean mixes than for rich mixes, again depending upon the type of air entraining agent used.

Air entrained concrete mixes that are designed to maintain cement content and strength by reducing the percentage of aggregate and the water-cement ratio are usually designated as "redesigned" mixes.

Air entrained concrete mixes, not so redesigned, are usually designated as "non-redesigned" mixes.

With no redesign of the mix, i.e., with identical water-cement ratio and reduced cement content, concrete compressive strengths are reduced 20-40% with 4-5% entrained air, depending upon the brand of cement, richness of mix, and type of air entraining agent used. According to work done by the Bureau of Reclamation on mixes containing $4\frac{1}{4}$ to $6\frac{3}{4}$ sacks of cement per cubic yard, concrete compressive strengths were on the average reduced $200\frac{7}{8}$ in² (5%) per per cent of entrained air when using an air entraining agent containing no catalyst or accelerator and $100\frac{7}{8}$ in² ($2\frac{1}{2}$ %) per per cent of air for an air entraining agent containing a catalyst (7). Greater or smaller reductions in strength can of course be obtained by using other air entraining agents.

Where mixes for air entrained concrete are redesigned, strengths of lean concretes containing 4-5% air will be increased from 0-40%, depending on the air entraining agent used, due substantially to the water cuts which can be taken in such redesigned lean mixes. However, for rich concretes, i.e., seven sacks per cubic yard or more, strengths will be reduced from 0-20%, again depending upon the air entraining agent used. As already indicated, this is due to two factors: (1) the larger percentage of air entraining agent required to entrain 4-5% air in such mixes, and (2) the smaller improvement in workability obtained with 4-5% air in rich concretes, which results in smaller water cuts being made possible. Fig. 10 gives a typical example of the effect of 4-5% entrained air on the compressive strengths of lean to rich concretes when using two types of air entraining agent, but the same cement throughout.

Curves B and C of Fig. 10 show relative compressive strengths for concretes containing two types of air entraining agents, the one for Curve C containing a catalyst and the one for Curve B containing neither a catalyst nor an accelerator. It is interesting to note that Curve C crosses Curve A at a cement content of $5\frac{1}{2}$ sacks per cubic yard, whereas Curve B crosses at 5 sacks per cubic yard. The use of still other air entraining agents might result in points of intersection of the relative strength curves at either higher or lower cement contents. In general, it is safe to assume that $5\frac{1}{2}$ sacks per cubic yard is the maximum cement content for which optimum results will be obtained upon the entrainment of 4-5% air. This maximum cement content will be lower for higher air contents.

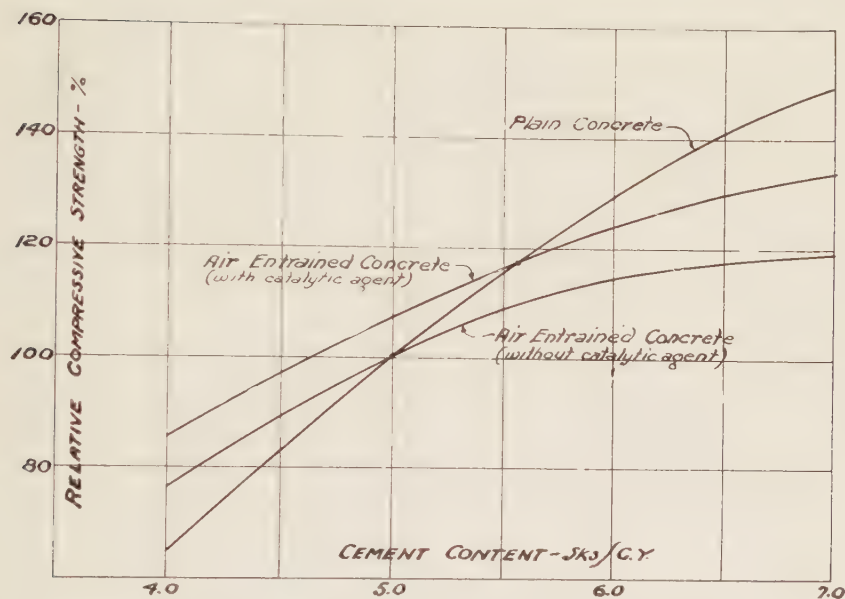


FIGURE 10 *Strength vs Cement Content for plain and air entrained Concrete, containing 4% air*

Fig. 10 also shows a trend of great importance in the comparison of lean and rich concretes, namely, the tendency of the air entrained concretes to develop little gain in compressive strength as cement content is increased beyond six sacks per cubic yard. It will be noted that the air entrained concrete containing the noncatalytic air entraining agent shows practically no gain in strength for cement contents between six and seven sacks per cubic yard. This phenomenon, which applies to flexural strengths as well as compressive strengths, accounts in large measure for the failure of redesigned rich mixes to regain in full the strength lost through the entrainment of air.

METHODS OF REDESIGNING AIR ENTRAINED CONCRETE MIXES

There are three general methods in common use today for redesigning concrete mixes containing entrained air. These methods are basically the same but differ slightly in their approach to the problem of compensating for the "bulking" of concrete due to entrained air. They are all predicated on the assumption that the percentage of air in the concrete is held between 3% and 5% of the volume of the

concrete, with an average of 4%. These methods are briefly characterized as follows:

1. American Concrete Institute Method

This method is the same as that used by the U. S. Bureau of Reclamation. Water content is arbitrarily reduced $8\frac{1}{2}$ lbs./cu. yd. for each one per cent of entrained air, and percentage of sand in the combined aggregate is reduced one per cent for each per cent air. The latter correction is made to eliminate the tendency of air entrained concretes containing normal sand percentage to act like sticky, oversanded mixes. Trial mixes are then made, and, if necessary, water adjusted further to maintain constant slump.

2. Public Roads Administration Method

In this method sand is reduced in the amount of 3% by weight of the combined aggregates. This reduction is constant for all mixes and apparently independent of the percentage of air entrained, but it assumes that 4% air is entrained on the average. Water reduction is not specified but is determined by trial mixes. For purposes of making the first trial mix, a water cut of 10% by weight can be assumed.

3. Goldbeck Method

In this method the absolute volume of sand is reduced to compensate exactly for the volume of air actually entrained. Sand reduction is therefore variable depending upon percentage of entrained air. Reduction of water is determined in the same way as in the Public Roads Administration method.

It should be borne in mind that in all of the above methods the amount of coarse aggregate or the amount of combined aggregate per cubic yard of concrete can be reduced instead of reducing the sand alone. The sand is generally eliminated in order to avoid oversanding of the mixes. However, in those cases where the normal mix contains a relatively low percentage of sand or where the sand is excessively deficient in fines, it may be found desirable to reduce the percentage of coarse aggregate. Job conditions must control the final decision in this matter.

In order to illustrate in detail the procedure for redesigning concrete mixes containing entrained air, the following sample calculation based on the American Concrete Institute method is given:

Given Mix (for concrete without entrained air)

Slump: 3" using 42% sand

Cement Content: $5\frac{1}{2}$ sks./cu. yd. = 517 lbs./cu. yd.

Water-Cement ratio: 7.0 gals./sk. = 0.62 by wt. = 320 lbs./cu. yd.

Mix Parts: 1 : 2.64 : 3.80 = 517 lbs. : 1365 lbs. : 1965 lbs.

Theor. Weight = $\frac{320 + 517 + 1365 + 1965}{27}$ = 154.3 lbs./cu. ft.

Assume:

4% air by volume of concrete to be entrained

Procedure: Mix redesigned to maintain slump and cement content

1. Reduce water 8.5#./cu. yd. for each per cent of air, or 34 lbs. for 4% air = 11% by wt. Then new water content = 320—34 = 286 lbs./cu. yd.
2. Reduce sand 23 lbs./cu. yd. for each 1% air or 92 lbs./cu. yd. for 4% air. Then, new sand weight = 1365—92 = 1273 lbs./cu. yd.
3. Make trial mix to see if water must be further adjusted to maintain three-inch slump. In this case, the trial mix indicated further adjustment was unnecessary. Hence
4. Final mix parts = 517 : 1273 : 1965 lbs./cu. yd.
5. Unit Weight =
$$\frac{286 + 517 + 1273 + 1965}{27} = 149.9\#/\text{cu. ft.}$$
6. Per cent Air =
$$\frac{154.3 - 149.9}{154.3} = 2.9$$

Item 6 under "Procedure" indicates air entrainment in the amount of 2.9 by volume of concrete, whereas 4% air was assumed. This discrepancy is due to the fact that the calculation was based on a comparison between plain and air entrained concretes of different designs, and therefore of different percentages of the concrete ingredients. In the case of the air entrained concrete, where substantial reductions in water and sand content were made, the unit weight of the resulting concrete exclusive of the air was greater. Hence, this method of calculating percentage of entrained air is inaccurate, but it is presented because it is frequently encountered in the field, due to the ease with which the calculation can be made.

It is therefore necessary for greater accuracy to calculate the theoretical weight of the air entrained concrete, utilizing the mix parts actually arrived at in the redesign. This check calculation is made as follows:

1. Absolute volume of the redesigned mix parts

$$\text{Cement } \frac{517}{3.15 \times 62.5} = 2.63 \text{ cu. ft.}$$

$$\text{Water } \frac{286}{7.5} = 3.81 \text{ cu. ft.}$$

$$\text{Sand } \frac{1273}{2.65 \times 62.5} = 7.70 \text{ cu. ft.}$$

$$\text{Stone } \frac{1965}{2.65 \times 62.5} = \frac{11.86}{26.00} \text{ cu. ft.}$$

2. Unit weight without entrained air

$$\frac{286 + 517 + 1273 + 1965}{26.0} = \frac{4041}{26.0} = 155.6 \text{ lbs./cu. ft.}$$
3. Unit weight with 4% entrained air

$$\frac{26.00}{0.96} = 27.1 \text{ cu. ft.} = \text{total volume with entrained air}$$

$$\frac{4041}{27.1} = 149.4 \text{ lbs./cu. ft.}$$
4. Per cent air actually entrained

$$\frac{155.6 - 149.4}{155.6} = 4.0\%$$

The above calculations show that the redesigned mix parts do not produce exactly 27 cu. ft. of concrete. They should, therefore, be adjusted accordingly. Also, it is possible that the desired slump will not be exactly obtained with the redesigned water-cement ratio. Therefore, trial batches should be made to determine slump and weight per cubic foot, and final adjustment of the mix parts based on the results of these trial tests.

EFFECT OF AIR ENTRAINING AGENTS ON OTHER PROPERTIES OF CONCRETE

The decreased water-cement ratios resulting from the greater workability imparted to concrete by entrained air and the bulking effect of the entrained air resulting in decreased settlement of the concrete in the plastic state cause a marked decrease in the tendency of the air entrained concrete to bleed. Therefore water gain, channeling, and sand streaking are correspondingly reduced. This makes for easier and quicker finishing of concrete surfaces after screeding or after form removal. The decreased water gain and channeling results in concrete of greater impermeability, a property which is still further improved by the greater density of the concrete matrix surrounding the bubbles of entrained air.

Low bleeding concretes are generally assumed to be more homogeneous in that there are smaller variations in water-cement ratio between the top and bottom of concrete pours. However, air entrained concretes do not appear to be homogeneous from the top to the bottom of a pour, despite the fact that they are low bleeders. This is indicated by abrasion tests made on the top, side, and bottom of

laboratory-prepared concrete specimens. Practically without exception, the resistance to abrasion of the concrete near the top of the pour is less than that at the bottom. For lack of a better explanation, it is postulated that this variation in resistance to abrasion between the top and bottom of a pour is due to migration of some of the entrained air with a consequent greater percentage of air in the top concrete.

Abrasion tests also show that resistance to abrasion is an inverse function of the percentage of air for percentages of air in excess of six per cent (8). Figure 11 shows abrasion specimens containing

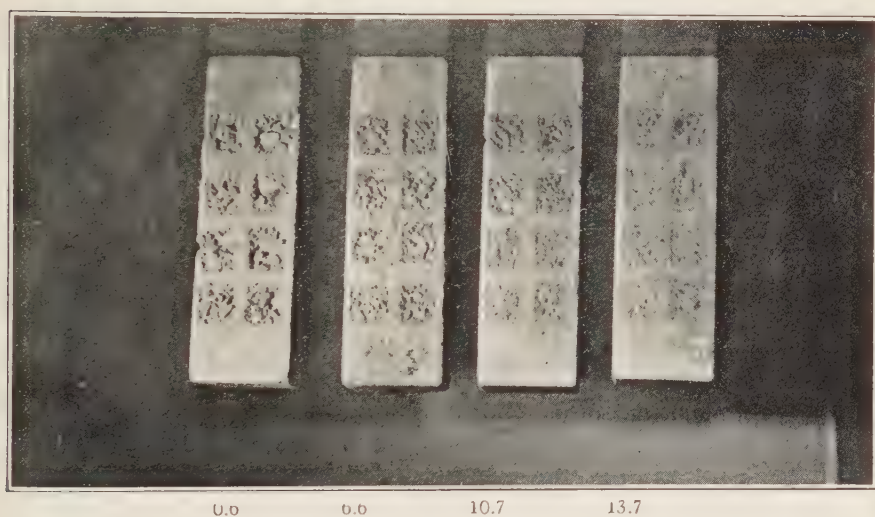


FIG. 11—PER CENT AIR.

various amounts of air which have been subjected for one minute to the abrasion effect of steel filings ejected from a nozzle placed $1\frac{1}{2}$ " from the specimen under an air pressure of sixty pounds per square inch. A steel template was used to confine the action of the filings to the $1\frac{1}{2}$ " squares shown. Each square represents one individual test or eight tests per specimen. Hence, the difference in weight of the specimen before and after being subjected to abrasion gives the total loss in weight for eight tests and should therefore be representative of the abrasion resistance of the concrete.

It will be noted that resistance to abrasion is seriously impaired by entrained air in excess of 6.6%, whereas the effect of 6.6% air

is negligible. It is therefore concluded that no serious loss in resistance to abrasion will be encountered if the air content is kept below six per cent as a maximum.

The same factors which contribute to nonhomogeneity of air entrained concrete also cause a reduction in the bond between concrete and reinforcing steel. Migrated entrained air trapped beneath horizontal reinforcing rods or bars reduces the contact area between the concrete and the steel, and thus reduces the total bond between the two. If the air entrained concrete mix is redesigned, the resulting decrease in water-cement ratio so increases the bond strength per unit of contact area that the loss in total bond is largely eliminated. Hence, if the mix is redesigned and if the entrained air is kept below six per cent, the reduction in total bond strength should never exceed ten per cent and in most cases will actually be practically zero.

Since resistance to freezing and thawing and chloride salt action is not the predominant factor in structural concrete, consideration of the effect of entrained air on resistance to abrasion and bond strength suggests the advisability of setting lower air content limits for structural concrete, say 2-4% instead of the average 3-5% called for by current pavement specifications. It is believed that these lower limits will make available to structural concrete most of the advantages of air entrainment without any corresponding detrimental effects.

ADDITIONS VS. ADMIXTURES

The foregoing has shown clearly the need for close control of the amount of entrained air in concrete. It is therefore desirable to know what factors govern the amount of air entrained in concrete by a given percentage of a given air entraining agent. It is generally conceded that the following factors are responsible for major variations in air content when the percentage of air entraining agent is held constant (6).

1. Richness of mix—lean mixes entrain more air than rich mixes. Percentage of air has been found in some cases to vary from $\frac{1}{2}$ to 8% when using an air entraining cement containing a given percentage of air entraining agent, cement content varying from four to seven sacks per cubic yard.

2. Percentage of sand—combined aggregates containing high percentages of sand will entrain more air than the same total aggregates containing lower percentages of sand. In one case on record, increasing the sand percentage from 38 to 42% by weight of total aggregate increased the entrained air from 4.0 to 5.5%.

3. Sand gradation—a deficiency in -30 to $+100$ mesh sand, or conversely an excess of $+30$ or -100 mesh sand will reduce the percentage of entrained air by as much as 3%.

4. Consistency—high slump concretes (up to 6 inches) will entrain more air than low slump concretes, all other conditions being the same. Increasing the slump from 2" to 4" will increase the entrained air approximately $1\frac{1}{2}\%$. The same slump increase in high slump concrete, say six-inch, will not affect the percentage of entrained air as much.

5. Temperature—temperature of the concrete as controlled by atmospheric temperature and temperature of the mixing water, aggregates, etc., has a slight effect on percentage of air entrained. The higher the temperature, the lower the percentage of air. A range of approximately 2% may be expected between temperatures of 40° F. and 90° F.

6. Length of mixing and type of mixer—large or small variations in air content may result from these variables, depending upon the type of mixer and type of air-entraining agent used.

7. Age of cement—where air entrainment is accomplished by the use of air entraining cement, age of the cement has been found to have a small effect on the amount of air entrained, depending upon the type of air entraining agent used.

The foregoing indicates clearly that the number of job variables affecting the air content of field concrete makes impossible the manufacture of an air entraining cement which will meet all possible job conditions (9). Furthermore, since the current A.S.T.M. specifications for Air Entraining Portland Cement were predicated on the use of this cement in low-slump, undersanded pavement mixes containing high cement contents, it is obvious that the use of such air entraining cements in structural concrete will necessarily cause the entrainment of excessive percentages of air in the concrete in place. As already stated, up to 8% air has been entrained in structural concrete when using an air entraining cement. For reasons, already given, this must be avoided at all costs.

For this reason it is the author's opinion that control of the air at the concrete mixer by use of a suitable admixture is the solution to the problem of maintaining the air content of concrete within specified limits. The principal objection to the use of admixtures is that it involves the addition of a fifth ingredient at the mixer with a consequent increase in the possibility of errors being made. As a result, too little or too much air entraining agent might be added, due to the personal equation. However, if too little air entraining agent is added, the resulting concrete would, under no circumstances, be worse than the nonair-entrained concrete used in the past. And the

use of an air entraining agent which is incapable of entraining in average concrete more than a predetermined maximum percentage of air goes far to counteract the possible addition of excess quantities of Air Entraining Agent.

The ultimate answer to control of the admixture at the mixer is the use of automatic dispensing or proportioning devices. There are currently available a fully automatic pump-type proportioner, for use on paving mixers, and a semi-automatic gravity-type proportioner for central concrete batching and mixing plants (10). There is in use today an inexpensive fully automatic, electrically timed proportioner for use in central batching and mixing plants, which is commercially available.

METHODS OF MEASURING ENTRAINED AIR

Fundamental to any discussion of control of air in concrete is a consideration of methods of measuring entrained air. The most commonly used method at the present time is the theoretically weight method. In this method a known volume, say $\frac{1}{2}$ or $\frac{1}{4}$ cu. ft., of air entrained concrete is weighed and the unit weight (pounds per cubic foot) calculated accordingly. The unit weight so determined is then compared to the theoretical unit weight of the same concrete, excluding the entrained air, calculation of which is based on a knowledge of the weights and specific gravities of the ingredients of the concrete. This method is illustrated in the sample calculation given previously in this paper.

Equally common is the gravimetric method for determining percentage of entrained air in concrete. In this method, the unit of weight of the concrete containing the air entraining agent is compared to the unit weight of similar concrete without the air entraining agent, both unit weights being determined by weighing known volumes of the respective green concretes. As already pointed out, this method is theoretically accurate only if the concrete containing no air entraining agent has the same mix proportions and same water-cement ratio as the concrete with the air entraining agent, but even if this provision is complied with, there arises the difficulty of placing the nonair-entrained concrete, resulting from the poor workability or plasticity of such concretes, and this in turn introduces a variation in percentage of air entrained, due to a marked variation in consistency. From a practical point of view, therefore, it should be

accepted that air determinations based upon the gravimetric method are never strictly accurate, and in some cases may deviate from the true value by as much as $1\frac{1}{2}\%$.

In both the theoretical weight and gravimetric methods, accurate knowledge of specific gravities of the ingredients and moisture content of the aggregates is essential, since calculated absolute volumes are based on these values. Any job variation in these values will materially affect the accuracy of the air content calculation. In one known case, random spot samples taken from stockpiles of a special type of aggregate were found to have specific gravities deviating by 15% from the average. Control of the air by these methods was obviously impossible in the case cited. In average concrete, using normal aggregates, less variations in specific gravity and moisture content occur, but such variations are sufficient in amount to affect the apparent air content by as much as 2%, unless, of course, frequent checks of the specific gravities and moisture content are made on the job and calculations revised accordingly.

In an attempt to eliminate gravity and moisture content as factors in the determination of air content, a picnometer method has been adapted to the measurement of entrained air by such agencies as the Indiana Highway Commission, the Michigan Highway Department, and the Portland Cement Association. In this method, a known volume of green concrete is stirred with a known volume of water, thereby diluting the concrete to such an extent that the entrained air is released and escapes to the surface. Measurement of the resulting reduction in combined volume is made by means of a hook gage and the volume of air released calculated accordingly. Results by this method have sometimes been erratic, due to incomplete release of the entrained air.

A fourth method, lately gaining prominence, is the Klein-Walker Air Meter method, in which air pressure is applied to a column of water enclosed above a known volume of concrete (11). The resulting change in the height of water in the column is noted. Since air is the only compressible material in the system so treated, the original volume of the air can be established by applying the pressure-volume relationship for a gas at constant temperature. Several modifications of the original Klein-Walker meter are currently being tried, applied air pressures varying from 30 psi to 4 psi. It is believed that low pressures are to be preferred, since particle interference of the aggre-

gates is thus minimized, and a modified meter along these lines has been developed by the Portland Cement Association. In this connection, it is interesting to note that the United States Engineer Corps is working on an air meter which will utilize vacuum instead of pressure, and by this means it is hoped that particle interference will be completely eliminated.

Comparison of data obtained by the air meter method and the theoretical weight method has thus far shown excellent correlation, provided that specific gravities of the ingredients were accurately known, so that the theoretical weight data used for comparison could be accepted as accurate and provided that a correction is applied to the air meter results to compensate for air absorbed by the aggregates when highly porous aggregates are used. Also, the air meter is portable and can easily be adapted to field use. Hence, it is felt that the air meter is the most promising of the methods currently available, and when test procedures for its operation are standardized may be the final answer to the problem of measurement of air in concrete.

SUMMARY

To sum up, it is believed that the following conclusions can be drawn regarding the beneficial effects of entrained air in portland cement concrete, based upon both laboratory and field experience and data:

1. Resistance of concrete to freezing and thawing and chloride salt action is greatly improved.
2. Workability of concrete in the plastic state is greatly improved and this results in easier placing, reduced segregation and better finishing.
3. Green settlement, hence bleeding, is reduced with a consequent improvement in the ease and speed of finishing.
4. Permeability is reduced, due to the greater density of the concrete matrix surrounding the microscopic air bubbles.
5. Resistance to the disintegrating action of sulfate waters, alkali, and sea water is increased due to the greater impermeability of air entrained concrete.

Also, in recapitulation the following conclusions regarding theory and precautions are made:

1. The plasticity imparted to portland cement concrete by the use of an air entraining addition or admixture depends upon the

amount and the grading of the fine aggregate used in the concrete. That entrained air does not depend for its plasticity effect upon the presence of portland cement is demonstrated by the tests which showed that a simulated concrete will have satisfactory workability or placeability even if no cement is used.

2. Consistent with the foregoing conclusions is the fact that optimum benefits can be derived from air entrainment only in relatively lean mixes. It is the author's belief that air entraining agents should not be used in concretes having cement contents of more than $6\frac{1}{2}$ sacks per cubic yard, if strengths must be maintained, unless air content is held at a low figure, i.e., 2-3%

3. The undesirable features of excessive air entrainment have been emphasized. Although excessive air will produce a marvelously plastic concrete, one that will have high resistance to freezing and thawing and salt action, air entrainment beyond six per cent by volume of the concrete is not recommended.

4. Since the need for improved resistance to freezing and thawing and salt action is less for structural concrete than for pavement concrete, it is suggested that the requisite improvements in the properties of concrete can be obtained by controlling the air content in such concrete within two to four per cent instead of three to six per cent limits currently imposed on pavement concrete.

5. For wintertime construction, the usual precautions for adequate curing and protection from freezing, and close control of the air content below four per cent are necessary in structural concrete.

6. Because of the great importance of controlling air content between definite specified limits, and because of the large number of variables affecting air content of concrete, control of the air by use of admixtures at the mixer are to be preferred. The practicability of the use of admixtures has been assured by the development of satisfactory automatic proportioning equipment for pavers and central mix equipment.

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GENERAL REMARKS ON THE DESIGN OF THE NEW JOHN HANCOCK BUILDING IN BOSTON

BY CHESTER N. GODFREY*

(Presented at a meeting of the Boston Society of Civil Engineers, November 20, 1946.)

THE insurance business is one of the important factors in the economy of our city. It is a safe assertion that the John Hancock is the largest of the several fine insurance companies in Boston.

The few comments that I shall present will serve to set before us the general features of the building, although it has already been well publicized and is doubtlessly familiar to many of you.

This is supposed to be a factual presentation. There is in Fig. 1



FIG. 1.—THE NEW JOHN HANCOCK BUILDING, AS IT WILL APPEAR FROM THE AIR.

*Executive Partner, Cram and Ferguson, Architects-Engineers.

a suggestion of nonfactualism. It serves, however, to illustrate how the completed building may appear from a plane's eye-view.

The main entrance is from Berkeley Street into the great lobby and the elevator lobbies. At the left of the entrance is a stadium-type auditorium, seating approximately 1200. This will at times be rented and used frequently for many purposes by the insurance company. There is rental space in the corner of Berkeley and St. James Avenue. Towards the rear on St. James is the Shipping and Receiving section. It will be noted that this is entirely within the building.

The typical bays are $20' \times 30'$. At the sides of the 8-story wings there is a 50' span extending the entire depth of the building.

The composition which you see in Fig. 2 is brought about by combining the company space requirements with the provisions of the city zoning laws in a design of utmost simplicity and strength. The building is larger than those that surround it but the low wings will harmonize this larger building with its neighbors and serve, as well, to buttress the great Hancock Tower that rears itself to the high-elevation in this area.

This is a large and high building. It should be emphasized that the directors of the company did not set about designing a monument. This building is the result of the very rapid growth of the company. Their space requirements are such that they must construct a building as large as is permissible under the law on their property.

The funds of the company are expended but sparsely for ornament, either on the exterior or the interior of the building. On the other hand, money is not spared on equipment that will contribute to the comfort and efficiency of the employees. The most modern arrangements for artificial illumination will be installed. The building will be air conditioned throughout and the basement rooms are as well ventilated as the rooms in the tower.

Vertical transportation is provided by means of escalators for floors from the basement to the eighth. The pair of escalators will be running up during the rush hours in the morning, and both of them down at night, while during the day they will alternate, one up and one down. There are to be 18 elevators serving all floors, principally those above the eighth.

We shall now briefly describe the uses of the special floors, passing over the many floors in the building that are devoted to the usual office arrangements, which are largely open areas with movable parti-



FIG. 2.—ARCHITECTURAL VIEW OF NEW JOHN HANCOCK BUILDING.

tions that will provide maximum flexibility for office arrangement and rearrangement.

The entire sub-basement area is largely taken up with equipment essential to a large building, such as air conditioning, electrical apparatus, shops, etc.

In the basement is located a small assembly room that may at times be rented in connection with the assembly hall above. There are conference rooms on this floor and a large area devoted to the shipping space, and a portion will be rented in connection with the banking room above.

On the sixth floor are located the kitchen and several small dining rooms. On the seventh floor is the principal dining room where approximately 4000 employees are served luncheon, in four shifts, each working day. The eighth floor is the employees' recreation floor with lounge rooms, library, the company store, etc.

The Directors' floor is the twentieth. The twenty-sixth story, Observation Floor, is to be glass enclosed on the four sides. It will at times be open to the public. From this floor, 355 feet above the sidewalk, will be a splendid view of Boston and surroundings. Above this floor, the area within the roof is used for elevator machinery, vent fans, cooling towers, etc. In the cupola above the main roof will be special lighting devices not yet completely determined.

Our consultant on special problems in connection with design of the steel frame is the well-known office of Jackson & Moreland. Few special problems have been encountered in the steel design. Our engineers are chiefly concerned with carrying out specific requirements of the Code.

The consultant on foundations is Dr. Casagrande. He has acted as something more than a consultant. Our engineers have designed these foundations under his initiative and direction.

THE PILE FOUNDATION FOR THE NEW JOHN HANCOCK BUILDING IN BOSTON

BY ARTHUR CASAGRANDE*

(Presented at a meeting of the Boston Society of Civil Engineers, November 20, 1946.)

INTRODUCTION

When Mr. Godfrey first asked me to assist in the design of the foundations for the new John Hancock Building, and displayed to me the architect's drawing for the new building (see Fig. 2 in preceding paper by C. N. Godfrey), my first reaction was a mixture of admiration for its beauty, and doubts about the possibility of founding such a heavy structure on the same type of floating foundation which has been successfully employed for the New England Mutual Building, and which the Architect-Engineers at first considered for this project. It seemed to me hardly possible to excavate the necessary great depth immediately adjacent to the existing John Hancock Building without resorting to very costly underpinning of this building.

The foundation design which was finally adopted aroused considerable interest and prompted the Boston Society of Civil Engineers to present this symposium on the design and construction of the foundations for this new building. In my contribution I am privileged to tell you about the studies which have led to the present design, and I shall present the results of observations to date which are being undertaken in part for construction control and in part for the purpose of providing necessary information to assist in the design and construction of any possible extension of this building in the future.

SUBSOIL CONDITIONS

The subsoil conditions at the site (illustrated in Fig. 1) are typical for large areas of Greater Boston, and particularly for the Back Bay area where the site is located. There is a surface stratum consisting of about 20 feet of fill which is largely sand and gravel. Beneath the fill one finds a layer of highly compressible organic silt

*Prof. of Soil Mechanics and Foundation Engineering, Graduate School of Engineering, Harvard University.

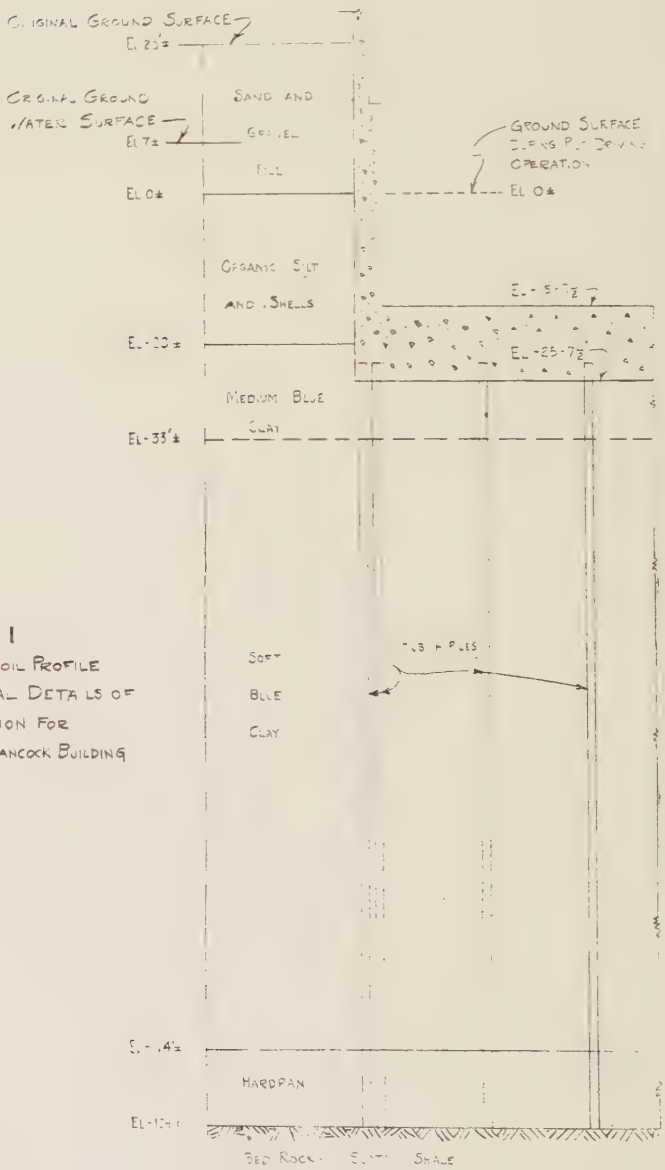


FIG. 1
AVERAGE SOIL PROFILE
WITH ESSENTIAL DETAILS OF
FOUNDATION FOR
THE NEW JOHN HANCOCK BUILDING

FIG. 1.

averaging 20 feet in thickness, and which is locally interspersed with layers or lenses of fine sand. Beneath the silt there is a thick stratum of so called Blue Clay for which the Boston foundation conditions are famous. The color is not really blue, but olive-greenish and grayish. Usually there is a top crust of stiff Blue or hard Yellow Clay which was formed by drying when the surface of the clay was exposed several thousand years ago. The soft clay is underlain by a thin stratum of very compact sand and gravel, which is generally referred to as hardpan. Bed rock consists of a slaty shale.

The customary foundation design in Boston for these soil conditions is to transfer the building loads by means of piles or concrete piers to the stiff crust which forms the upper layer of the clay stratum.

Most of the older buildings are relatively light, so that the weight of soil removed by the ordinary basement depth is almost equal to the weight of the building. Hence, such buildings do not create an important net increase of stress in the clay and there are no settlements to speak of. However, when the weight of a building is considerably greater than the weight of soil removed, then the clay has to carry additional load to which it will adjust itself by gradually decreasing its volume, or in other words, the clay will consolidate under the additional load.

SETTLEMENTS OF BUILDINGS ON BOSTON BLUE CLAY*

The beneficial effect of deep basements is clearly demonstrated by the comparison of settlements of large buildings in the Back Bay area shown in Fig. 2 which have all similar subsoil conditions.

The largest settlements, curve S, have occurred on a building which is only about one half of a basement deep below original ground surface. Ten years after construction the maximum settlement was 15 inches. Today it is almost 20 inches.

Another building in the Back Bay Area, a hotel built early in this century, has practically no basement excavation and its settlement curve follows closely the curve for building S. Its maximum settlement today is more than two feet.

Curves C and T are for buildings which extend one full basement below original ground surface, a greater depth of excavation than

*For more detailed information see the following references:

R. E. Fadum, "Observations and Analysis of Building Settlements in Boston". Doctor's thesis, Harvard Graduate School of Engineering, May 1941.

A. Casagrande and R. E. Fadum, "Application of Soil Mechanics in Designing Building Foundations". Transactions A.S.C.E., 1944, pp. 399-415 and 463-478.

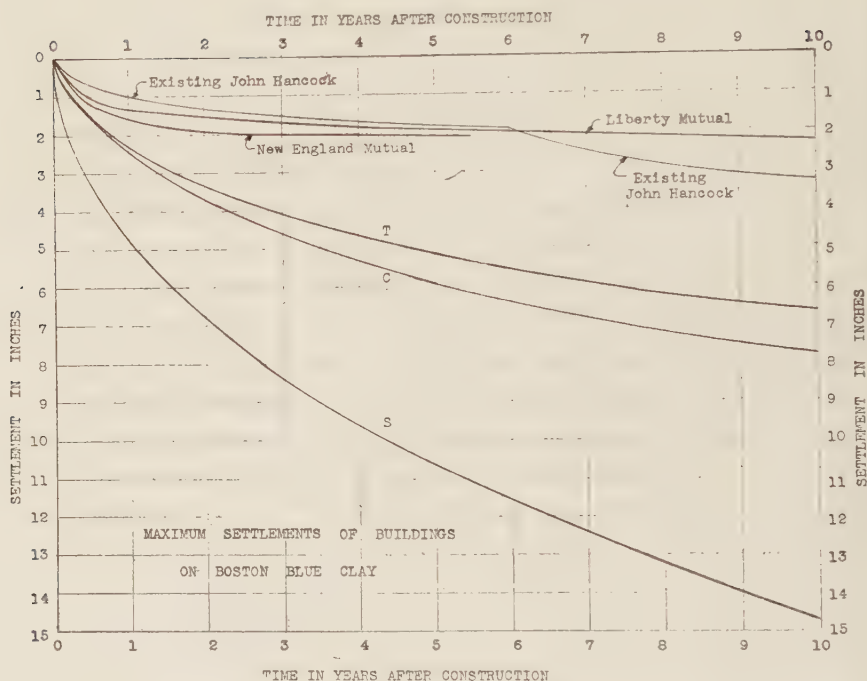


FIG. 2.—A COMPARISON BETWEEN MAXIMUM SETTLEMENTS OF SEVEN LARGE BUILDINGS RESTING ON BOSTON BLUE CLAY.

for building S or the hotel referred to above. The settlements of buildings C and T are correspondingly smaller.

Contrasted to these buildings, the existing John Hancock Building has a depth of excavation of almost two basements below original ground surface. In fact, as originally built, the weight of the excavation was about equal to the weight of the building, and as a result the settlements were very slight as shown by the uppermost curve. About six years after the original construction, this building was enlarged, causing a considerable increase in the net load on the soft clay. As a result, the settlements increased, although they are still of a tolerable magnitude.

The curve which starts out just below the curve for the John Hancock building shows the maximum settlements of the Liberty Mutual Insurance Building. In the foundation design of this building a deliberate attempt was made to reduce the settlements by controlling the depth of excavation. The total load of the building is

only slightly more than the weight of excavated material, and therefore the total settlements are very modest. The maximum settlement to date is only a little over two inches.

Slightly below this curve runs the curve of maximum settlement for the New England Mutual Life Insurance Building. The total weight of excavation of this building, as it stands now, is actually larger than the weight of the building itself. As a result, the long-time settlements due to consolidation of the clay are entirely eliminated, as can be seen by the very flat slope of the settlement curve. If we take into account the present rate of settling of the entire Back Bay area, which is in part due to the weight of artificial fill and in part due to the general lowering of the ground water table, one arrives at the paradoxical result that this building is now actually rising slightly in relation to the original ground surface. The sharp initial settlement is primarily the re-compression of the clay which follows the heave as developed during the excavation. The maximum heave was of the order of three inches, and the maximum settlement to date only slightly over 2 inches.

Fig. 3 shows for the same buildings the maximum differential

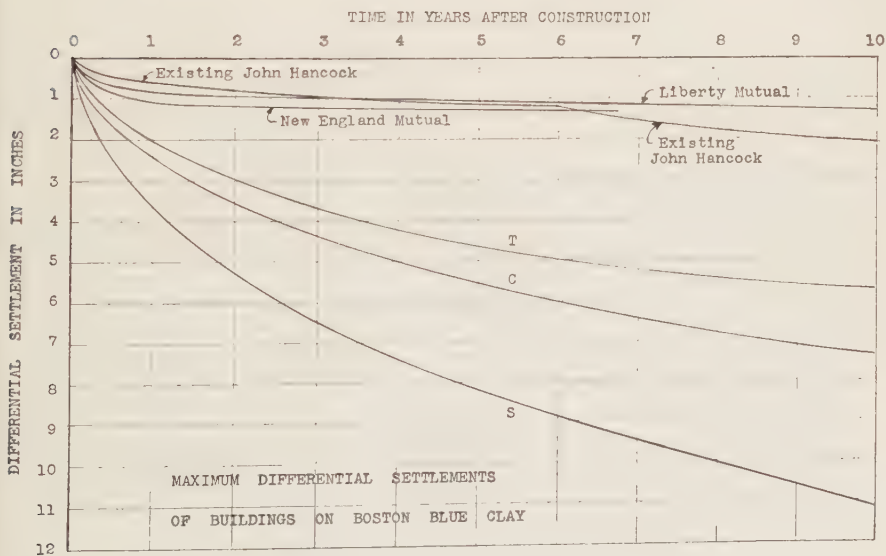


FIG. 3.—A COMPARISON BETWEEN MAXIMUM DIFFERENTIAL SETTLEMENTS OF SEVEN LARGE BUILDINGS RESTING ON BOSTON BLUE CLAY.

settlements, that is the maximum differences in elevation which developed within each building. These maximum differential settlements are usually more significant from the standpoint of damage to the buildings than the maximum settlements.

For curves S, C, and T these differential settlements are of objectionable magnitude and have caused considerable cracking. However, for the existing John Hancock Building, the maximum differential settlement at the end of ten years was only 2 inches, and to date is only $2\frac{1}{2}$ inches, a tolerable magnitude which has not resulted in any cracks.

For the Liberty and New England Mutual Buildings, the maximum differential settlements to date are about $1\frac{3}{8}$ inches. The differential settlements for the Liberty Mutual Building are still increasing slightly, but for the New England Mutual Building there has been practically no change during the past two years in spite of the very large differences in loading between different portions of that building. This satisfactory performance of the New England Mutual foundation is due to the combination of a deep excavation (two basements) and the stiffness provided by a system of 40 ft high girders.¹

DESIGN OF FOUNDATION FOR THE NEW JOHN HANCOCK BUILDING

From this brief review it can be seen that the principle of floating foundation design, particularly if combined with a stiff girder foundation for redistribution of unequal loading, does permit the construction of monumental buildings on the soft Boston Blue Clay which will undergo only small and tolerable settlements.

In view of the success of the foundations of the New England Mutual Building which the writer designed for Cram and Ferguson, the Architect-Engineers who also designed the new John Hancock Building, it was only reasonable that at first one contemplated the application of the same principles to the design of the foundations for the new John Hancock Building. However, the difficulties which would accompany the great depth of excavation necessary to float this building, and the strongly expressed desire of the owners to adopt a design which would reduce to a minimum all risk to neighboring structures, including their own existing building, led me to recommend that the loads be transferred to bed rock.

¹For a detailed discussion of these foundations the reader is referred to the Transactions A.S.C.E., 1944, pages 406-415 and 485-488.

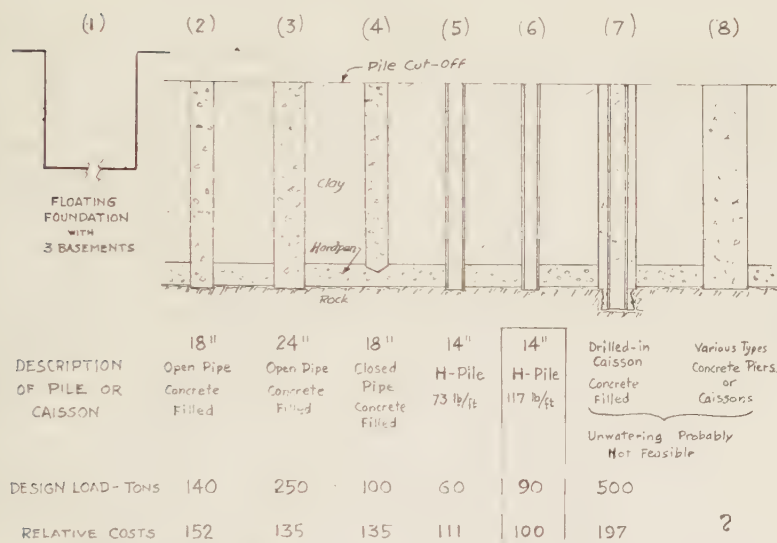


FIG. 4.—TYPES OF FOUNDATIONS AND THEIR RELATIVE ESTIMATED COSTS CONSIDERED FOR THE NEW JOHN HANCOCK BUILDING.

Among the methods for transferring the loads to rock, those shown in Fig. 4, No. 2 to 8, were carefully considered. First we investigated the possibility of driving 18 inch or 24 inch diameter steel pipes open-ended, and filled with concrete, as they were used at the Mystic Power Plant of the Boston Edison Company. It was realized that in the restricted area of the site the procedure of cleaning out the pipe with compressed air and water, and the removal of the material which was blown from the pipes, would be an awkward operation, to be avoided if at all possible. To overcome these objections, the suggestion was made to drive 18 inch closed-end pipes into the hardpan, reducing the volume displacement as much as possible by pre-excitation. However, the proposed procedure would still have left a substantial amount of clay to be displaced by the closed-end pipes, which would have resulted undoubtedly in serious heaving of the adjacent portion of the existing John Hancock Building.

At that juncture I recommended the use of steel H-piles, because I believed that they would give a minimum of trouble due to soil displacement. I did it reluctantly because to my knowledge no monumental building of this character had ever been built on such piles. Their chief use has been in the construction of trestles, foundations

for bridges and industrial plants, and waterfront structures of which we have a good example in Boston in the reconstruction of the Commonwealth Pier.

The resistance against use of H-piles for foundations of monumental buildings can be traced to the following doubts:

First, the question of the corrosion of steel in the ground. Under certain soil conditions steel cannot be considered a permanent type of construction. At the John Hancock site the only soil which may cause corrosion of steel is the organic silt-clay overlying the inorganic clay. In the absence of any clear-cut evidence one way or the other, it was fortunate that this question could be eliminated by the simple expedient of excavating the additional few feet of organic silt which remained below the bottom of the slab, and by thickening the slab by that amount. It was first proposed to replace the additional depth of excavation by sand and gravel fill, but the Builder expressed the opinion that the additional thickness of concrete would not be much more expensive, considering the question of proper placement and compaction of sand and gravel fill within the restricted area of the small pockets in which the foundation slab will be placed. Whatever questions might be raised about the economy of the foundation slab, this solution certainly simplified the problem of corrosion of the piles, since they are now embedded only in concrete and in the directly underlying, chemically inert foundation clay. Placing the foundation slab directly on the clay has the additional advantage that any ground water movement around the piles will be entirely prevented, since for practical purposes there is no ground water flow within the clay.

Mr. Irving B. Crosby, consulting geologist, who has a thorough knowledge of local geology, was engaged to make a special study regarding the corrosive effect of the foundation clay on steel. He concluded that the steel piles would have an indefinite life if entirely embedded in this clay and the underlying hardpan and rock.

The second doubt which is often expressed by engineers, is that of the danger of buckling of long, slender steel H-piles when surrounded by a soft soil. The great majority of the load tests on H-piles have been made in such a manner that the load was allowed to remain on the pile only a relatively short period. Therefore, such tests do not prove the safety against buckling under long-time loading, when the pile is embedded in soft soil of a type which could yield plastically.

There are a few load tests on H-piles embedded in very soft material where the load was permitted to remain for a sufficient length of time to test the safety against buckling. From a paper by G. G. Greulich* I cite the following examples: (1) For the Potomac River Bridge at Ludlow's Ferry, Maryland, 215 foot long, 14", 102 lb ft H-piles were driven through 90 feet of soft river mud and below that through various other soils, to refusal at a depth of about 190 feet. The mud was so soft that the piles sank 77 feet under their own weight, and a total of almost 100 feet with the weight of the hammer added. A test load of 200 tons was allowed to remain constant for a period of six weeks, to determine whether a total length of 114 feet of pile standing in air, water and this soft mud, would show any tendency to buckle. There were no such signs. (2) In Houston, Texas, 12", 72 lb ft H-piles, 90 feet long, were driven through 60 feet of soft clay, 12 feet of hard clay, and the tops of the piles standing in water and air and exposed to the swaying of the piles in a pier. A test pile was loaded with 100 tons and this load allowed to remain for one year without any signs of buckling.

Mr. O. G. Julian of Jackson and Moreland, consultants on this project, made a theoretical analysis of the stability of the proposed H-piles, assuming a very conservative lateral resistance for the soft Blue Clay. Mr. Julian's analysis showed that there is no possibility of buckling of the H-piles under the design loads proposed for this project, even if a pile should be bowed as much as one foot. It is conceivable that due to obstructions such as boulders, some of the piles might be bent much worse. Even then, such piles could still carry a very substantial load and they would automatically transfer any additional load that might cause buckling, onto the surrounding piles. This is an important advantage of any pile foundation utilizing many piles. In contrast, foundations of a type in which an entire column load is resting on one single pile, require the utmost assurance as to quality of installation of each pile and perfect embedment in hard bed rock, if serious trouble is to be avoided.

It would not be telling the truth if I were to say that my recommendation of a steel H-pile foundation was readily accepted by all those who coöperated in the design of this project. However, after the various studies were completed the only doubt that lingered on was

*G. G. Greulich: "Design Loads for Steel H-Beam Bearing Piles"; presented at the 27th Annual Meeting of the American Association of State Highway Officials, September 30, 1941.

the fact that such foundations had not yet been accepted by engineering practice as suitable for monumental buildings of this type. At this point I might comment that it is not the first time that Boston engineers have departed from generally accepted foundation practice and have pioneered new ideas. However, one word of warning would be in order. Once a precedent is established, it happens only too often that others follow more thoughtlessly. It may well be that the important question of corrosion of steel in the ground may not continue to receive the deserved attention. It would be of real help if the Boston Building Department or some other interested organization would initiate long-time observations of corrosion of steel in the organic silt overlying the clay, so that we may eventually have reliable information for the design of foundations in which the piles may be partially embedded in the organic silt.

Various methods of sinking concrete piers to rock were also considered. Because of the water-bearing sand and gravel overlying rock and the fact that in the borings objectional water conditions were encountered in the rock itself, it was concluded that these piers would at best be difficult to install. We have also considered the so-called drilled-in-caisson. Because of said water conditions it was concluded that most of these caissons would not be accessible for inspection. Furthermore, for the relatively soft rock we believed that the maximum load we could dare to place on one of these caissons should not exceed 500 tons. Besides, even this load would have to be proven to the satisfaction of the Boston Building Department by load tests carried to twice the proposed design load.

For the various types of piles the following design loads were assumed as a basis for a comparative cost analysis:

- 140 tons on 18" open pipes, driven to rock and concrete filled.
- 250 tons on 24" open pipes, driven to rock and concrete filled.
- 100 tons on 18" closed pipes, concrete filled, driven into hardpan.
- 60 tons on 14", 73 lb/ft H-piles, driven to rock.
- 90 tons on 14", 117 lb/ft H-piles, driven to rock.

The relative costs of the various feasible foundations are shown at the bottom of Fig. 4, based on the cost figure of 100 for the 14", 117 lb/ft H-pile foundation. No cost estimates were made for the various types of concrete piers, since experienced contractors had ruled this procedure out as impractical because of the water conditions. The relative costs represent the averages of at least three

independent estimates by experienced engineers who differed greatly among themselves. There is no question that this is the most controversial subject in my talk tonight. Nevertheless, I believe that there is little doubt that the type of H-pile chosen for this foundation is the most economical type of pile for this project. This result of the cost studies made it easier to have the H-pile foundation accepted by all concerned. However, I should like to emphasize that it was not due to consideration of cost that I recommended the H-pile foundation long before the cost studies were started, but because of my desire to employ a pile which would result in a minimum of soil displacement.

PILE LOAD TESTS

Because of the requirements of the Boston Building Code it was necessary to prove the proposed design loads by means of pile load tests carried to at least twice the proposed working load for which a value of 90 tons had been tentatively adopted.

Although it was decided to use 117 lb/ft, 14" H-piles, only 73 lb/ft, 14" H-piles were available on short notice. Therefore, these lighter piles were used for the load tests. Obviously, if we could prove that a 14" H-pile weighing only 73 lb/ft is safe for a load of 90 tons, then this load should be even safer for a 60 per cent heavier 14" H-pile.

It was desired to utilize the driving record of these test piles also for the purpose of deriving suitable specifications for the final resistance to which these piles should be driven. So as not to distort the driving record due to obstructions in the granular fill, Gow cylinders were sunk through this fill to the surface of the organic silt and the test piles were driven inside the cylinders.

The two test piles were driven with a No. 0 Vulcan hammer, in sections of about 65' length each, and spliced by welding in the leads. They penetrated through the organic silt with practically no resistance. Through the Blue Clay the resistance was fairly uniform, and as soon as the piles struck the compact sand and gravel, or hardpan, overlying the bedrock, the resistance began to increase rapidly. Driving was stopped when the resistance reached about 60 blows for the last six inches. As far as it is possible to determine by comparison with the profiles of adjacent borings, both test piles were driven to bearing on bedrock.

The load tests were made by means of a water-ballast loading

equipment having a capacity of slightly over 200 tons. Four large steel tanks were resting on a platform which in turn was balanced (somewhat precariously) on the pile itself. The weight of the platform and tanks was known, and the variable weight of the water in the tanks was controlled and measured by means of calibrated stand pipes.

On the first of the two piles to be tested the deformation of a bearing plate resting on top of the pile caused the load to be concentrated in the corners of the pile. This, together with some eccentricity in the load on the platform, caused a high concentration of load on the corners of the pile and buckling of one flange when the load reached 140 tons. The load was removed, the damaged section cut off, and the loading repeated. However, the same flange buckled again, this time under a load of 180 tons.

To prevent overstressing of the corners of test pile No. 2, an 11 inch square plate was placed between the two large bearing plates on top of the pile. In this manner the bending of the plates on top of the pile was prevented and a more uniform distribution of the load over the cross-section of the pile assured. After this change no difficulties were encountered in loading the second pile to the capacity of the loading equipment. The 200 ton load was allowed to remain constant for a period of 51 hours and then the load was reduced to 100 tons, which was allowed to remain as long as the construction activities permitted, a total of 42 days.

In Fig. 5 is plotted the movement of the top of test pile No. 2. The straight line represents the theoretical compression of the pile, assuming that the entire load is transferred to rock and that the rock is incompressible. The slightly curved line above the straight line is the actual movement of the top of the pile as measured by the extensometers. Under the 100 ton load, which remained constant for 51 hours, there was only a slight time effect. The 200 ton load was also kept constant for 51 hours, and it showed a more substantial compression with time which is shown in Fig. 6. During the rebound the load was maintained constant at 100 tons for 42 days, during which period, apparently, a substantial recovery of the compression took place. However, it was later discovered that probably most of this was movement of the bench marks to which the measurements were referred.* When finally all load was removed, a permanent settlement of the top of the pile of about $\frac{1}{8}$ of an inch remained.

*The bench marks were located near the piles and were subject to slight fluctuations of the ground surface due to natural fluctuations of the ground water table. Even minor seasonal fluctuations of the ground water surface cause measurable vertical movements of the ground surface because of the highly compressible character of the underlying organic silt.

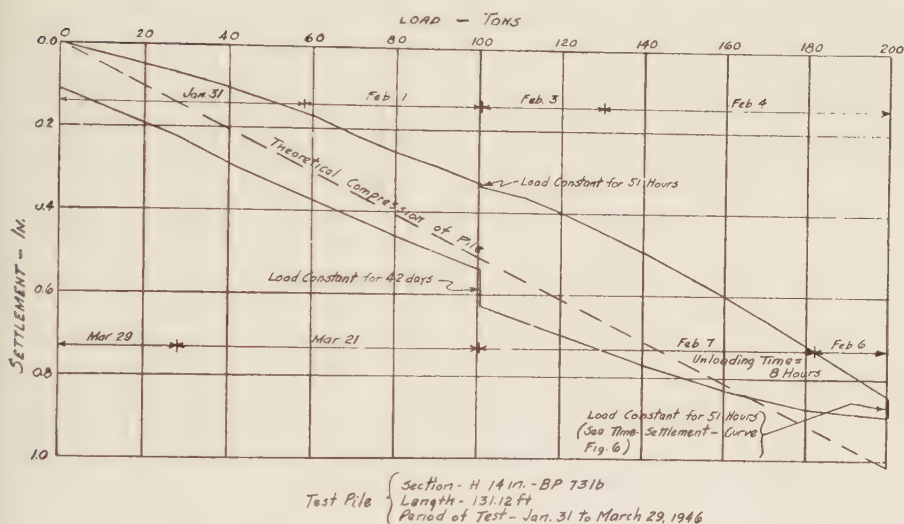


FIG. 5.—LOAD SETTLEMENT CURVE FOR TEST PILE NO. 2.

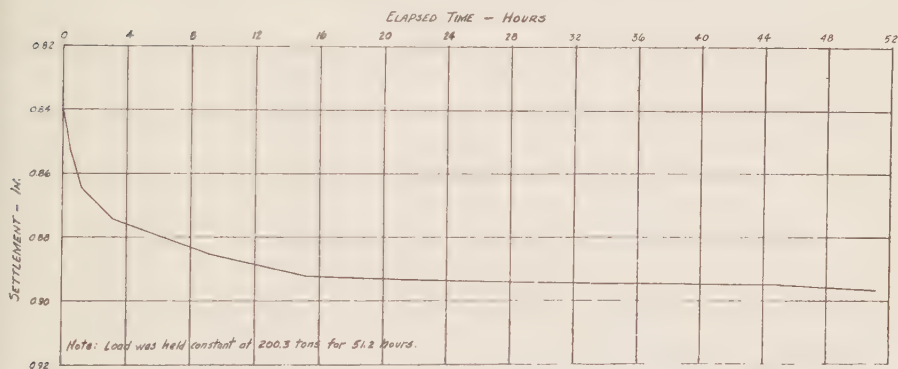


FIG. 6.—TIME EFFECT UNDER 200 TON LOAD FOR TEST PILE NO. 2.

The movement of the top of the pile is less than the theoretical compression of the pile only. Therefore, it is concluded that during the loading a portion of the load is transmitted by skin friction into the surrounding clay. Since during unloading the rebound curve is also flatter than the theoretical elastic expansion of the pile, skin friction must have prevented the pile from developing its full elastic expansion. The exact proportion of the load which reaches bed rock cannot be determined because the compression of the rock is an un-

known quantity. I estimate that of the maximum load of 200 tons, at least 100 tons were actually transmitted to bed rock and into the layer of hardpan immediately overlying it. Because of the almost complete recovery of the compression it follows that this load did not cause overstressing of the rock and hardpan.

From these test results it was concluded that the proposed working load of 90 tons on 117 lb, 14" H-piles would be a very conservative loading.

It was also concluded that the final resistance to which these test piles were driven would form a suitable initial specification for the driving of the piles, but subject to change as the work progressed. In the actual installation of the piles a heavier hammer was used, yielding about 25 per cent more energy per blow than the No. 0 Vulcan hammer. It was found that 60 blows for the last 6 inches with this heavier hammer gave a satisfactory assurance of reaching bed rock, yet without danger of overdriving. In this connection it is pertinent to note that in the past such piles have generally been driven to a much greater resistance. In my opinion this is not only a waste of time and energy but such overdriving may cause serious damage to the piles. I believe that the manner in which piles are driven on this project is heavy enough to insure proper contact with the bearing stratum and yet light enough to protect the piles against damage.

OBSERVATIONS ON THE HEAVE OF THE CLAY SURFACE

As will be explained by Mr. Parsons in his contribution to this symposium, excavation is progressing in two stages. During the first stage the entire site was excavated to a depth of about 20 ft. From this general excavation level all piles are driven. Then the deep excavation will proceed in small pockets and the foundation slab in each pocket will be completed before an adjacent pocket is excavated. It is believed that this procedure will provide insurance against objectionable settlements of the existing John Hancock Building as well as against heaving of the piles due to being pulled up by the expansion of the unloaded foundation clay.

To measure the heave of the clay surface which is caused first by unloading the clay due to excavation and later due to the volume displacement of the piles, two observation points were installed in the top of the clay near the middle of the building site. In Fig. 7 is shown the progress of the observed vertical movement plotted as the



Fig. 7
COMPARISON OF VERTICAL MOVEMENT
OF UPPER SURFACE OF CLAY WITH
CUMULATIVE RECORD OF PILE DRIVING

Fig 9
VERTICAL MOVEMENT OF
A TYPICAL PILE DRIVEN TO ROCK
AND OF ADJACENT SURFACE OF CLAY

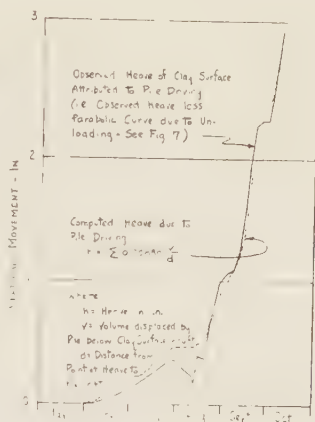


Fig 8
OBSERVED HEAVE OF CLAY SURFACE
ATTRIBUTED TO PILE DRIVING
COMPARED WITH COMPUTED HEAVE
DUE TO PILE DRIVING



average curve for the two observation points. For comparison are also shown in this figure a theoretical heaving curve due to unloading by excavation, and a cumulative curve of the number of piles driven.

In the period from March to June 1946 the heave was obviously expansion of the clay due to unloading by the general excavation to a depth of about 20 ft. If pile driving had not been started in June, the heave would have continued probably along the dashed parabolic curve representing theoretical expansion due to unloading.

When pile driving was started in June, the surface of the clay began to heave more rapidly due to the volume displacement of the piles. Theoretically, if all of the displaced volume of all the piles would be distributed over the building area, and none outside the area, it should cause a uniform heave of 8 inches. One should expect that in the center of the building site the heaving would be approximately equal to the heave so computed. However, the measured heave was found to progress at only about $\frac{2}{3}$ the rate of this theoretical heave, until about 60 per cent of the piles were driven. While the last third of the piles is being driven, a comparison cannot be made because deep excavation and concreting of the foundation slab have started, which introduce other variables.

Comparison of the curve showing the number of piles installed with the heave of the clay surface, Fig. 7, shows a fairly good agreement. A much better agreement is achieved by taking into account the distance of the piles from the observation points. Assuming that the effect decreases with the first power of this distance, one obtains the theoretical heave curve due to the displacement of the piles shown in Fig. 8, the shape of which agrees well with the observed heave curve.

During August there was a period of about 20 days in which practically no pile driving was done. One should expect during this period to observe only the slight rate of heave due to swelling of the clay caused by the unloading. However, instead of a rise, a rather sharp settlement of the observation points took place. From this the tentative conclusion was drawn that the slight deformation of the clay due to the installation of piles has disturbed the clay structure sufficiently to initiate some consolidation. The fact that the total heave is considerably less than what was expected due to the displacement of the piles is probably also due to this tendency of the clay to consolidate as a result of the disturbance of its structure caused by the pile driving operations.

Summarizing the causes for the movement of the clay surface, we have found that it is the result of three different effects: first, the heave due to the swelling of the clay caused by the reduction in load due to excavation; second, the heave due to soil displacement by the piles; third, settlement due to consolidation of the clay resulting from disturbance to the clay structure by pile driving. Soil Mechanics is not sufficiently advanced to be able to predict what the net result of these three influences will be. The third effect will probably have important implications regarding the design of the foundation slab. The consolidation of the clay resulting from disturbance to the clay structure may mean that only little or no swelling pressures will develop against the bottom of the foundation slab. This is an important question on which reliable information is urgently needed. Therefore, it was decided to install into the bottom of the slab a number of earth pressure cells which will allow us to measure the soil pressure.

OBSERVATIONS ON THE HEAVE OF THE PILES

Another important series of observations are those on the movement of the piles themselves after they are installed.

Most of the piles were driven in two steps. The first one consisted in driving the piles from the operating level, which is about 20 feet above cut-off. The second operation consisted in driving the piles with a follower to rock.

The vertical movements were observed on piles that were partially driven as well as on piles completely driven to rock. To enable observations on piles driven to rock, a number of them were driven with extra length so that they would reach rock without the use of a follower. This permitted observations directly on the top of such piles. In addition, observations were made directly on the top of the 200 lb piles (see next heading) driven without a follower adjacent to the existing building.

Observations were made on the heaving of 11 partially driven piles, while other piles were being driven in the immediate vicinity. In a period of 19 days the heaving amounted to an average of 1 and $\frac{3}{16}$ inches, with a maximum of 1 and $\frac{3}{4}$ and a minimum of $\frac{1}{2}$ inch.

In Fig. 9 is shown a typical heaving record of a pile driven with extra length to rock, along with observations of the heaving of the immediately adjacent clay surface. This pile, as well as the other

piles which were driven with extra length to rock, remained practically unaffected by the heave of the surrounding clay. The surface of the clay immediately adjacent to this pile heaved, under the effect of the driving of surrounding piles, about 3 inches in one month as shown in Fig. 9.

As illustrated by Fig. 9, the piles are not affected by the heave of the surrounding clay once they are driven to rock. I admit that before the results of these observations became available I had felt some concern that the heaving of the clay might lift the piles sufficiently to necessitate re-driving after final excavation.

SPECIAL PILES ADJACENT TO EXISTING JOHN HANCOCK BUILDING

As an additional precaution against objectionable settlements of the east wall of the existing John Hancock Building, thirty-three 33" 200 lb WF-beams were driven in line with the adjacent foundation wall of the new building. The purpose of these special piles is to reduce the lateral deformation of the foundation clay which will tend to develop as a result of deep excavation on one side, and of the load of the existing building on the other side of the boundary line between the existing and the new building.

At this time the east wall of the existing building has not yet settled, but due to the displacement of soil by the piles a slight heaving has developed.

SETTLEMENT OBSERVATIONS OF SURROUNDING STREETS AND BUILDINGS

Frequent levels are run on the surrounding streets and buildings to check the effects of the construction operations. Very accurate settlement observations are being made in the basement of the Liberty Mutual Insurance Company. Since construction of the new John Hancock Building was started in the spring of 1946, to the end of October 1946, the west wall of the Liberty Mutual Building has settled about 1/16" more than what the normal settlement for this period would have been as indicated by past settlement observations on this building.

CLOSURE AND ACKNOWLEDGMENT

It is hoped to supplement this paper after completion of the new building by an account of the results of the many observations

which are being undertaken. For example, it will be of interest to the profession to learn of the foundation pressures which will be measured by means of pressure cells installed in the bottom of the foundation slab. The main purpose of these cells is to measure the swelling pressure of the clay, if any, against the massive concrete slab which is held rigidly in place by the numerous steel piles.

Many of the measurements and observations on this project will prove of value and interest to the designers of other large buildings in the Boston Back Bay area.

Progress in the application of soil mechanics to the design and construction of foundations and earth works is still largely dependent on accurate observations. Unfortunately on the great majority of projects the needed observations are either poorly made, or not made at all, particularly if the success of the project appears assured. When something does go wrong, it is usually too late for the measurements which would have given the necessary information for deciding on appropriate corrective measures. Fortunately, all organizations connected with this project—the John Hancock Mutual Life Insurance Company, the architect-engineers Cram and Ferguson, and the Turner Construction Company—have all demonstrated an excellent spirit of cooperation when faced with the many, and what sometimes must have appeared annoying, requests by the soil mechanics consultant for more accurate and a greater number of observations.

I should also like to use this opportunity to thank Mr. H. A. Mohr, of the Raymond Concrete Pile Company which conducted the pile tests and installed all steel piles, to whose sound, practical advice this foundation project also owes a great deal. I could well apply to him the saying that there is no limit to what a man can accomplish if he does not care who gets the credit.

Finally, my best thanks to Mr. Stuart B. Avery Jr., who has ably assisted me in the task of collecting and interpreting the data which are here presented.

CONSTRUCTION PROCEDURE FOR THE FOUNDATION OF THE NEW JOHN HANCOCK BUILDING

BY M. H. PARSONS*

(Presented at a meeting of the Boston Society of Civil Engineers, November 20, 1946.)

THE excavation for the foundation of the John Hancock Mutual Life Insurance Company Building is being handled in two stages: the first stage from street elevation to approximately El. +1, and the second stage from El. +1 to El. —25.6. We were prompted to handle the job in this manner so as to give the maximum stability to the surrounding streets and structures by not unloading the entire area at one time, and to provide sufficient support for the pile driving equipment by not excavating all of the gravel fill. This fill extends in the Back Bay area from the existing street elevation of approximately +20 to the bottom of the old bay at approximately El. zero to —1. Below this elevation is a silt formation extending to approximately El. —25, where the borings indicate firm clay.

Ground water elevation in the general area at the start of the excavation was approximately at El. +5.5. In order to de-water the building site during shovel excavation, three caisson shells were installed to the elevation of the top of the silt, and these were provided with pumps. As excavation proceeded, it was a simple matter to lead the water in trenches to these pump locations.

As the digging continued to El. +1 over the entire lot, the banks were sloped by hand to an angle of repose, and a ditch at the foot was installed to collect the ground water on top of the silt formation. After 12", 53 lb/ft H bracing piles were driven on the wall line around the perimeter of the building, as shown in Fig. 1, an 8" perforated pipe was installed at the top of the silt layer. This pipe was buried in gravel and drained on St. James Avenue in a westerly direction and on Berkeley Street in a southerly direction. Water control on Stuart Street and the south end of Berkeley Street became a different problem, due to the fact that the top of the underlying silt layer was in some instances at El. —8, that is, at a considerably great-

*Project Manager, Turner Construction Company.

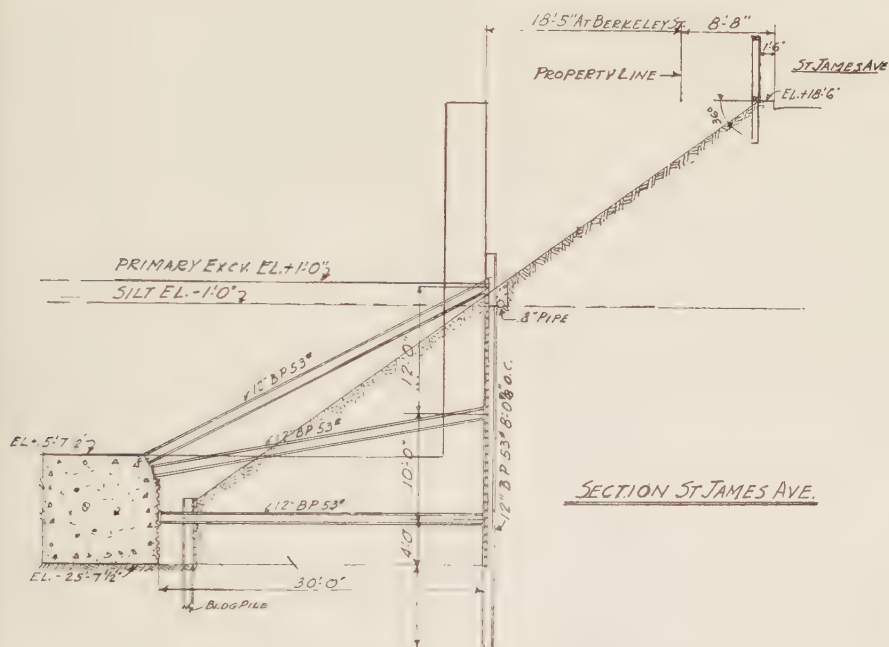


FIG. 1.—ELEVATION OF TYPICAL BRACING SYSTEM.

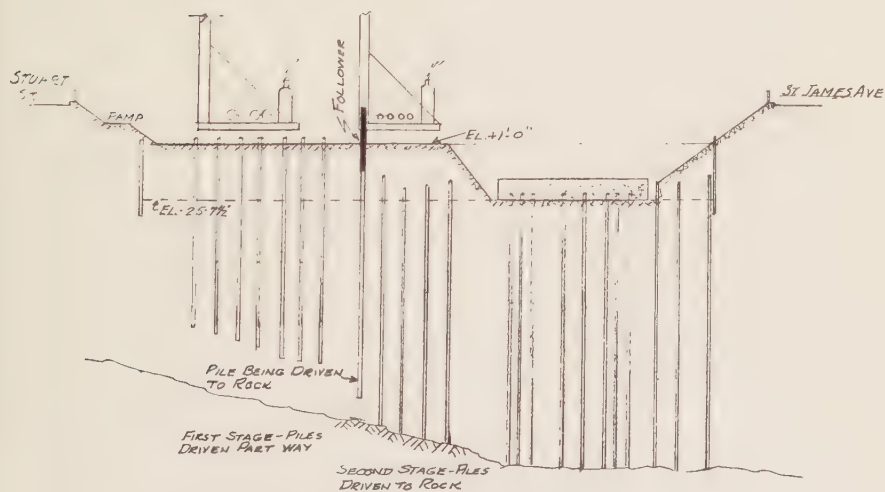


FIG. 2.—CROSS SECTION OF BUILDING SITE ILLUSTRATING STAGES OF PILE DRIVING OPERATIONS.

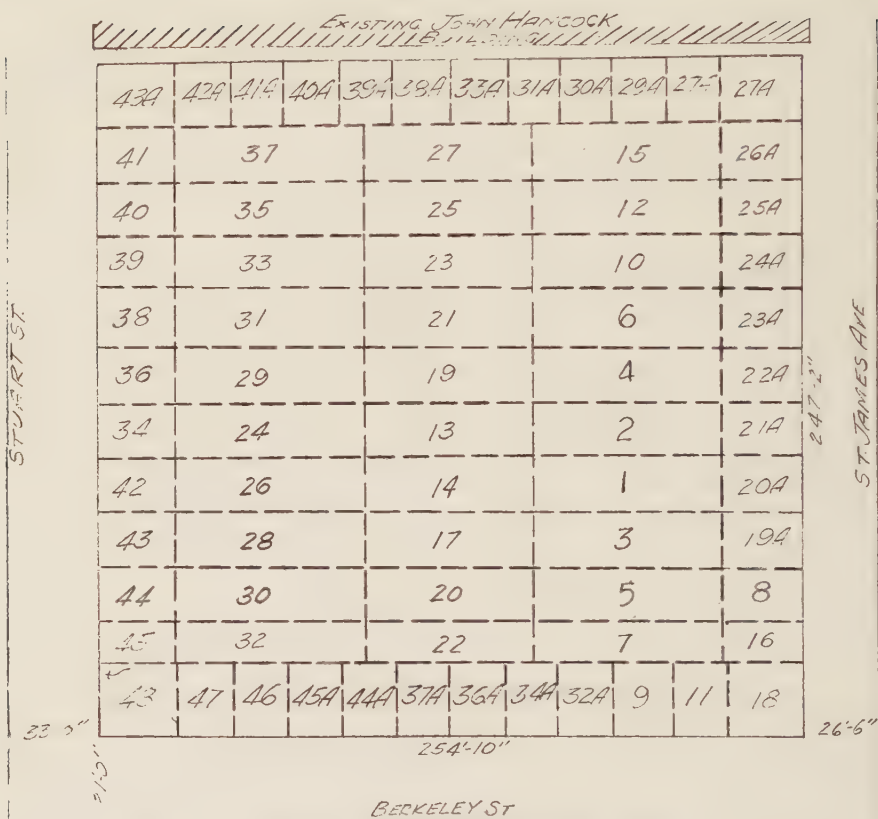


FIG. 3.—PLAN OF FOUNDATION SLAB SHOWING PROPOSED CONSTRUCTION SEQUENCE.

er depth than was desirable to excavate at the time. It was, therefore, decided to install well points in these locations. The silt on the west side of the lot was at El. —3, and drainage was taken from Stuart Street to the St. James Avenue pumps through an 8" perforated pipe.

Pile driving operations were started on the corner of Berkeley Street and St. James Avenue. The 14", 117 lb/ft piles were driven in one section, after first being spliced at the assembly yard in Charlestown. Driving was accomplished in two stages: first, by driving the 120' piles so that their tops were approximately flush with the ground at El. +1, and then by "following" these piles with a second and smaller driving rig down to the required resistance on rock, as shown in Fig. 2. The necessary pile lengths had been previously determined by interpolation from core borings.

Final excavation is being accomplished in sections of approximately $20' \times 60'$, each section being concreted before the firm clay in the adjacent section is uncovered. Thus, it is possible to replace the load on the firm clay surface in such a manner as to minimize any possible swelling or uplift of the firm blue clay. Fig. 3 shows the proposed sequence of this concreting operation for the foundation slab. The areas immediately adjacent to the exterior building wall are to be excavated after the center portion of the mat is well under way. This permits the center mat to take the load of the banks through diagonal bracing, shown in Fig. 1, while this excavation is in progress. Horizontal 4" timber lagging is being installed from El. +1 downward as the excavation progresses. This lagging will be supported by the 12", 53 lb ft piles previously installed, as indicated on Fig. 1.

In order to remove the excavation and supply sufficient concrete and reinforcing in the amount of time allotted, a precast concrete.

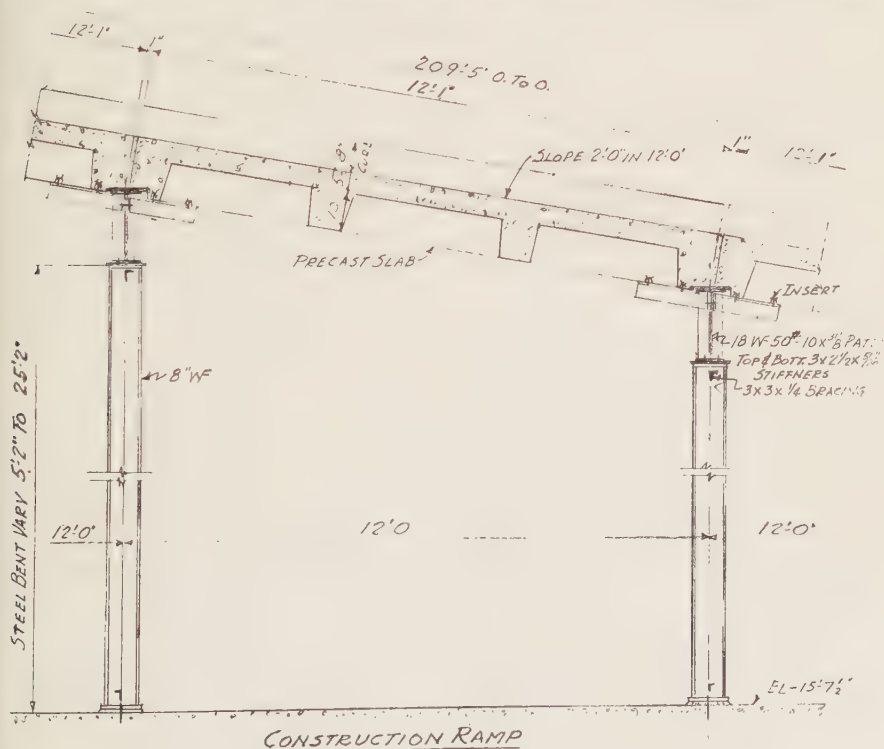


FIG. 4.—TYPICAL SECTION OF PRECAST CONCRETE RAMP.

two-way ramp will be installed on the first ten mat slabs. This ramp will permit the abandonment of the earth ramp on Stuart Street, which must be removed as excavation proceeds. The ramp will be supported on structural steel bents as shown in Fig. 4. It is designed to permit two concrete trucks or a power shovel to pass freely without any additional precautions.

The 10' concrete foundation slab is being formed by the use of steel sections of Robertson flooring. The corrugations in these forms provide sufficient keying action to develop the shear required in the construction joints, as shown in Fig. 5. These forms were made in units large enough to permit erection in a four-hour period and thus assure a rapid sequence of operations, so desirable in the construction of a foundation of this character.

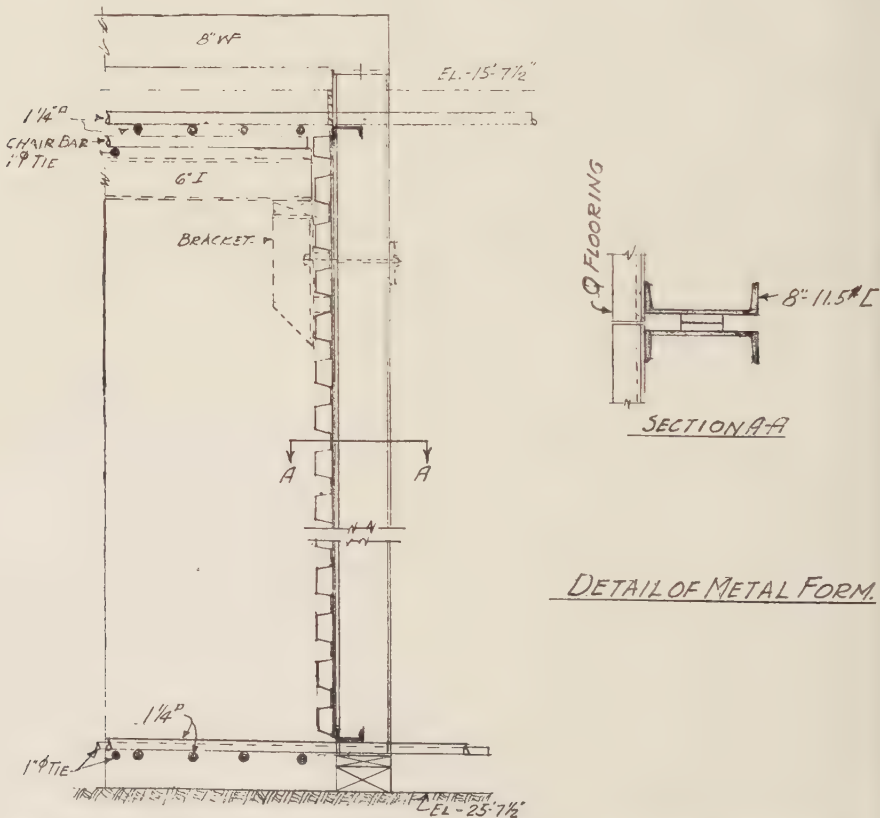


FIG. 5.—TYPICAL CROSS SECTION THROUGH A FORM FOR CONCRETE FOUNDATION SLAB.

REPAIRS TO DAM AT SOUTH BARRE, MASSACHUSETTS

BY HOWARD M. TURNER*

(Presented at a meeting of the Hydraulics Section, B.S.C.E., held on May 7, 1947.)

I HAVE found that sometimes small repair or rebuilding jobs, particularly water jobs, have as much and sometimes more engineering interest than new larger work. This is partly because it is impossible to determine in advance what special conditions are going to be found in existing structures. Then, too, a small job does not permit the use of large and expensive methods and equipment which may be possible on a big one. This requires an adaptability of the methods used on the work both of design and construction. I found the repairs to the dam of the Barre Wool Combing Company on the Ware River at South Barre a job of this kind.

The present dam at Barre was built about 1904. It was certainly the second dam at the site and may be the third or fourth. It consisted of a spillway of stone filled timber crib about 95 ft. long with non-overflow section on the north end of stone masonry with fill upstream 200 ft. long. The dam is not on ledge. In 1935, this old dam was entirely rebuilt. A concrete ogee spillway section was placed just below the timber crib dam, the concrete being poured right against the timber and then carried over the top raising the level 3 ft. Part of the non-overflow section was also made into a spillway by means of a concrete ogee section both downstream from the masonry and over its top, the elevation of this portion being 4 ft. higher than the other part of the new spillway. Stone paving was placed below the spillway toe. New abutments were built on each side to a height of 7 ft. above the north half of the dam and 11 ft. above the south half. A heavy fill was put in extending approximately 50 ft. upstream from the dam with its elevation sloping gently down from the crest. On the south end there is a head gate and intake structure built in 1914 for a 10 ft. penstock which leads down to a hydro-electric station. Process water for the mill is taken from this penstock. Similar methods of repairing old log dams are not uncommon. The timber dam

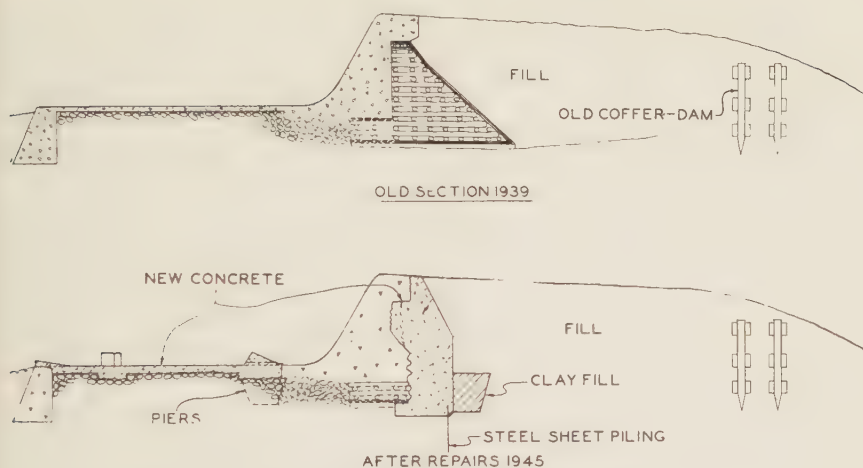
*Consulting Engineer and Professor of the Practice of Engineering, Harvard University.

of the Wheelwright Company on the Ware River has a heavy ogee apron of concrete built below it. This differs from that, however, in the way the concrete is carried over the top of the old dam and in the heavy fill on the upstream deck of the wooden crib.

The 1936 flood did some damage below the dam requiring some repairs. A new concrete apron was put in the dam extending downstream 30 ft. from the toe and the walls on each side were extended downstream. The 1938 flood undermined these walls and washed out holes below the new apron which required its reconstruction. The holes below the apron were filled in with a heavy paving. It was proposed at that time that the apron should later be extended downstream but this work was never done.

In the summer of 1944 when the water was drawn down below the crest of the dam it was apparent that there were leaks through the upstream fill, mostly about 6 or 7 ft. back from the concrete crest. These were of sufficient size in some cases so that, when the water was pulled down nearly to the level where the leak occurred, a small whirlpool could be seen. At other places it was not so evident but the appearance of the top of the gravel fill showed that water had been seeping through. Various attempts were made during that season to stop these leaks but without apparent success. It was decided, however, not to attempt under existing conditions to do more than try and locate the worst leaks and fill them in. After the winter, however, it was found that conditions were becoming much worse and water was beginning to appear bubbling up downstream from the concrete toe of the dam in the joint between the dam and the apron. Further explorations were made to try and trace the leaks to see if they could be stopped but, as this was apparently not going to yield any success, it was decided to do a real repair job on this portion of the dam. It was pretty clear that the planking of the old timber dam which gave the structure its main water tightness was failing.

The plan adopted consisted of the removal of enough of the timber dam so that adequate width of river bottom would be exposed to provide a suitable cut-off and allow the construction of a sufficient block of concrete behind the original concrete portion of the dam to create a stable concrete dam. The final plan adopted is shown in Fig. 1 which gives a typical cross section. Borings showed the existence of hardpan below the dam at a distance within reach. The new section was carried down to this hardpan. A concrete cut-off was at



CROSS SECTION OF DAM AT SOUTH BARRE, MASS.

FIG. 1.

first proposed but it was found that steel sheet piling could be driven to sufficient depth to provide an adequate cut-off. No attempt was made to drive this piling below a depth of more than 6 ft, but most of it was successfully driven to that depth. As the old dam was exposed it was found that it was in very bad condition and there was no question but what anything less than complete repairs would have been ineffective.

There was some question before the bottom of the old dam was finally reached as to just what was under the upstream portion of the original concrete part. Inquiries from people who had been there during construction were conflicting. It was finally found, however, as far as could be determined from the upstream side, that this concrete rested on a stone filled timber crib which presumably formed an apron below the original dam. It was decided to leave this crib in on the basis that the timbers would always be wet. The new concrete was allowed to run in among the timbers as far as possible.

In computing the stability of this design it was assumed that for the water seeping under the dam, head was lost lineally with the length of flow, the path of flow being taken from the top of the earth fill around the clay fill and the cut-off to the log crib under the dam. On this basis the resultant, with maximum water over the crest, came at about the center of the distance between the upstream heel of the

new concrete and the downstream toe of the old concrete section. Actually the timber crib below the dam underneath the old concrete section almost certainly acts as a drain to relieve upward pressure from that point downstream. This was shown by the necessity of pumping downstream from the old dam through holes excavated in the concrete apron to prevent backwater flow into the construction. The appearance of some of these holes seemed to show that in places during the time when the leaks were coming out below the toe of the dam, some fine material had been washed from the gravel on which the concrete was placed. In order to consolidate this gravel without interfering with its drainage possibilities, five piers were constructed of concrete below the toe of the old dam down to the hardpan foundation. These piers were approximately 3 ft. square at the bottom and were filled with concrete which was allowed to run into the surrounding gravel. They were spaced about 18.5 ft. It was not intended that these should take all the load but that they should reinforce possible weak spots in the gravel foundation under the dam above the hardpan.

The work was done during the summer of 1945. The water above the dam was drawn down by the sluice gate on the north half of the dam. A coffer dam was built across in front of the head gates which were closed, process water being taken into the penstock through a 24-in pipe which extended through the coffer dam into the pond above. The two ends of the dam required some special measures, particularly the south end where it was necessary to get an adequate tight connection of the new concrete with the headgate structure. This necessitated underpinning the abutment. Fig. 2 is a photograph of the construction.

It was clear from past experience that precautions against erosion at the toe during floods were required. The Ware River at Barre has a drainage area of about 104 sq. mi. The Metropolitan District Commission's diversion at Coldbrook where the drainage area is 96.8 sq. mi. is only 2 miles above this point. This diversion takes the "flood" waters in excess of about 131 c.f.s., but the maximum it can take is limited to about 3,000 c.f.s. which is not a very large proportion of a large flood so that it cannot be considered in any way as flood control. The 1936 flood had a peak of 6,800 c.f.s., 65 c.f.s. per sq. mi. The 1938 flood had a peak of 15,000 c.f.s., 144 c.f.s. per sq. mi. Both of these came after the diversion at Coldbrook. The proposed Barre Falls flood control reservoir controlling 57 sq. mi. when built, will reduce the floods at this dam.

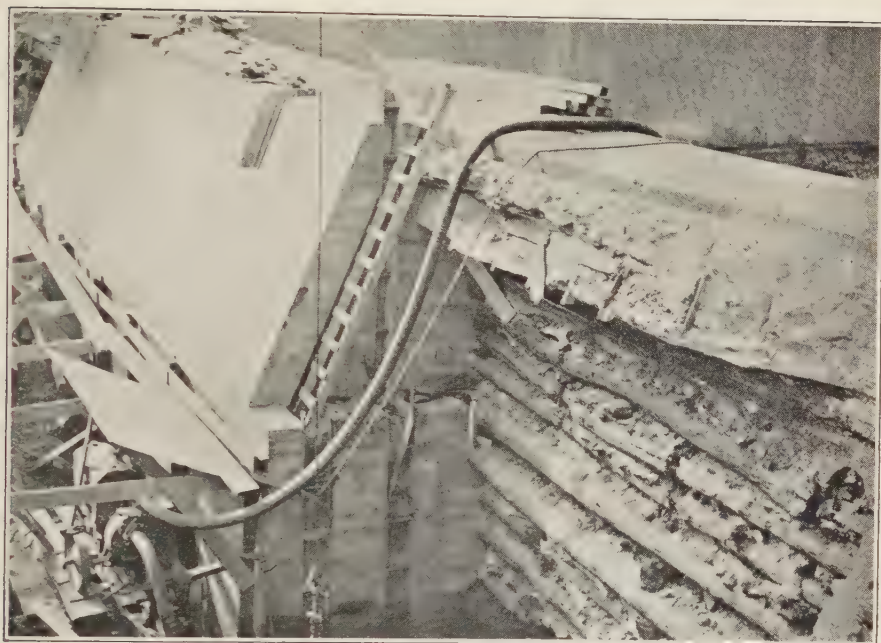
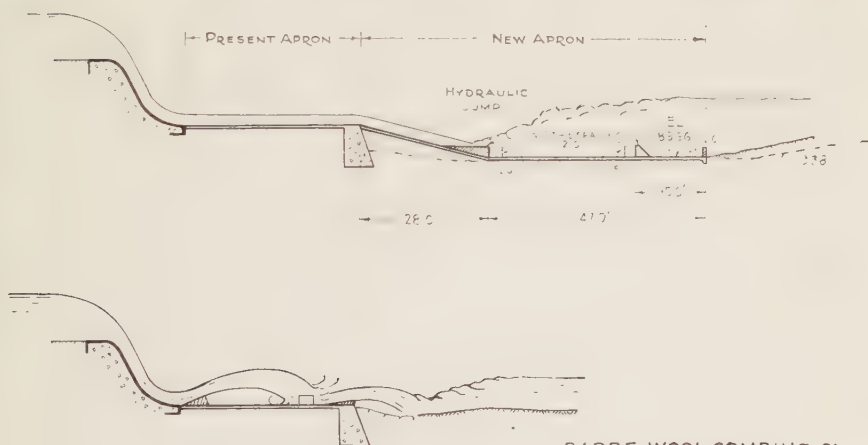


FIG. 2.



BARRE WOOL COMBING CO.
TOE OF DAM

Howard M. Turner
Cons. Engr. Boston, Mass.

FIG. 3.

The concrete apron extending 30 ft. below the dam, which was put in to replace the stone paving washed out by the 1936 flood, was badly damaged in the 1938 flood. It was replaced and a heavy toe wall was built all the way across the river. It was proposed at that time this apron should be carried downstream an additional 90 ft. to the end of the abutment wall on the south bank. Instead of this, heavy stone paving was put in immediately below the new wall. Experience during the years after this work was done showed that successive floods even of moderate size would wash out this stone paving. The question of what to do to prevent this erosion of the toe of this dam became a part of the design.

A study was made of the possibility of obtaining a hydraulic jump on the apron or immediately below the apron. The profile of the 1938 flood below the dam was not available but the flood height at the bridge across the river 550 ft. downstream was known so that a slope could be approximated. It was found that the depth was too shallow to obtain a hydraulic jump on the apron under these flood conditions. It was clear that extending the apron downstream at the same level was not a solution. It was possible to obtain such a jump by excavating below the dam but this meant lowering the river bed 7.4 ft. and extending this low elevation for 78.3 ft. to take care of the required length for the jump.

Even with an arrangement of baffles and toe walls as proposed by the Bureau of Reclamation standards, this lowered section of the river bed below the dam would have had to be 5.0 ft. below the level of the concrete apron with a total length at river level of 75 ft. This is shown on Fig. 3. It is interesting to note that during the 1938 flood the river level was washed down pretty nearly to this point in a portion of the river bed. This hole was filled in to support the stone paving below the concrete wall at the end of the apron.

This seemed very expensive and unsatisfactory from any point of view and it was decided to investigate some form of bucket or baffles which would destroy the energy sufficiently so that the water would not wash below the existing apron. Experiments were conducted for this purpose at the Alden Hydraulic Laboratory at Worcester. The final design consists of a slight kick-up bucket at the bottom of the spillway which throws the water up into the air and a series of baffle piers placed on the apron at the point where the stream of water from this kick-up again hits the apron. There is also a slight lift to the apron at its very downstream edge. See Fig. 3.

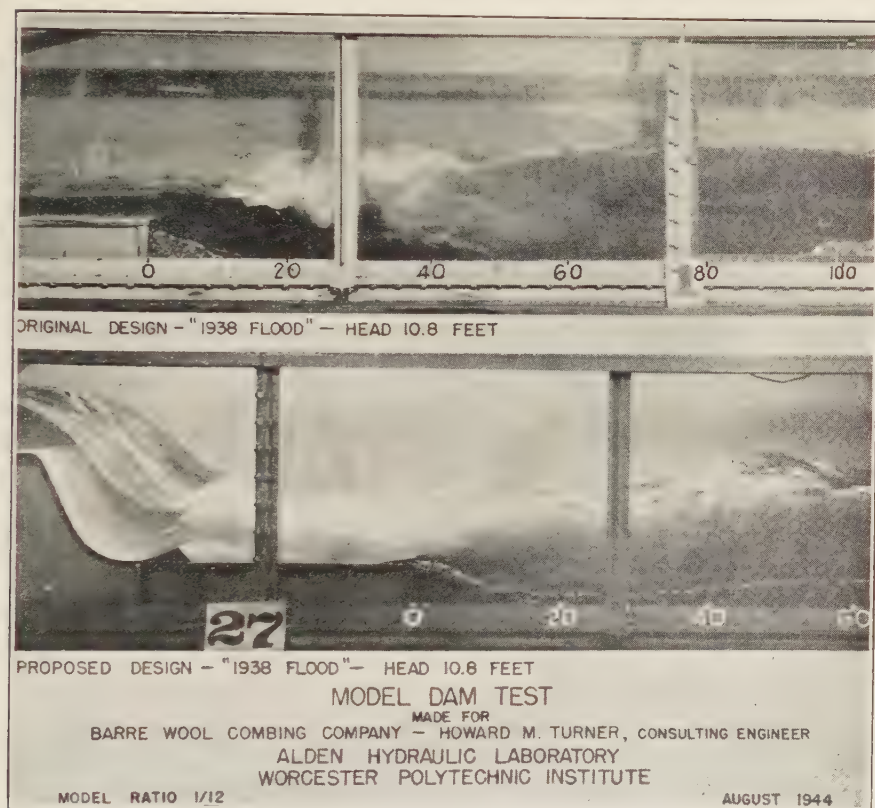
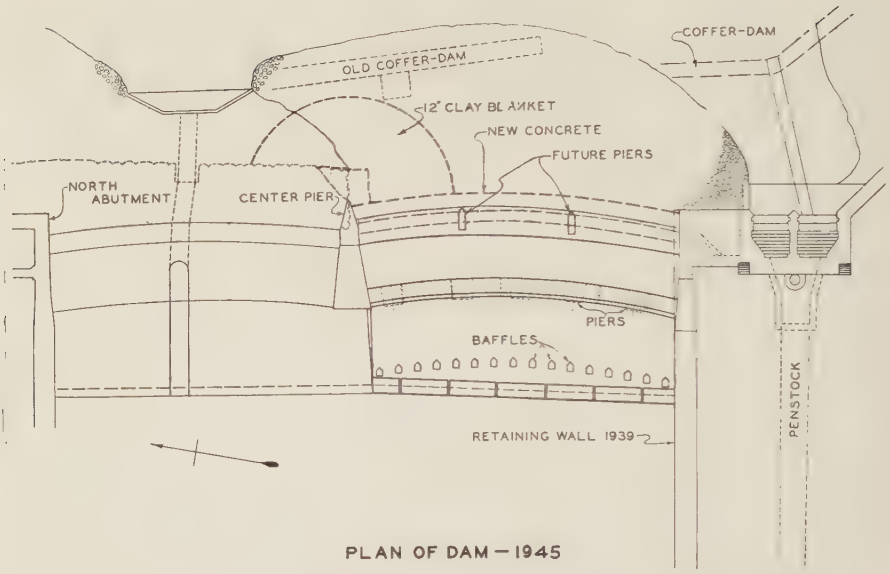


FIG. 4.

The photograph Fig. 4 shows the comparison between the original condition which swept all the bottom of the "river" out of the flume for a distance equivalent to 100 ft. downstream from the apron and the final design which left the stone filling only slightly below the level of the apron.

The model was made with a scale of 1-inch to the foot. The quantity of water was controlled in the flume above the dam. The tail water level below the dam was controlled by a raising or lowering of a gate hinged at the bottom. The elevations above the dam and in the tail water were computed for various floods so that for any desired flood discharge it was only necessary to adjust the head water level to give the required amount of water over the crest and then adjust the tail water gate until the proper tail level was secured.



PLAN OF DAM - 1945

FIG. 5.

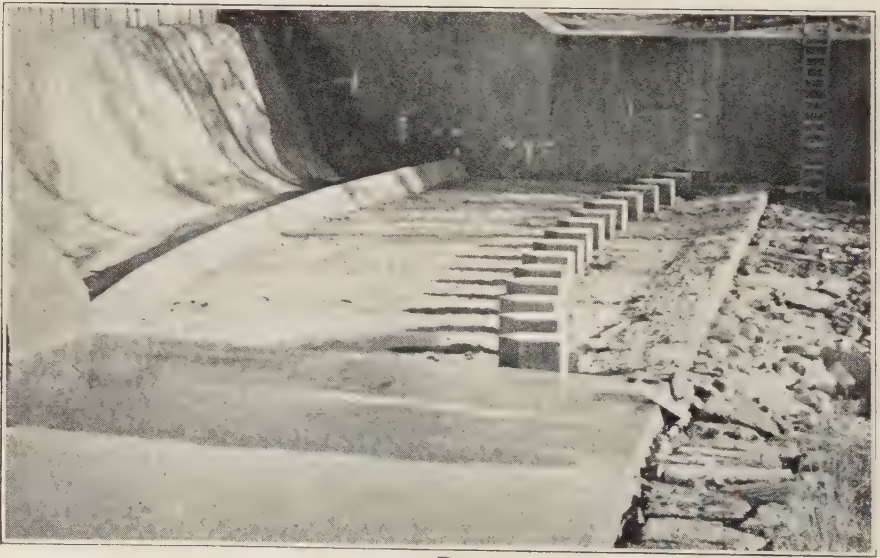


FIG. 6.

In general, two floods were tried for each case. One, 10.8 ft. above the crest substantially equivalent to the 1938 flood, the other 6.7 ft. equivalent to the 1936 flood. Various steps were taken, briefly as follows: first a kick-up at the bottom of the dam but with no baffles. Baffles were then added, first across below the kick-up, then on the kick-up itself and finally at or near the final location. Fig. 5 shows a plan and Fig. 6 a picture of the apron as it was finally constructed.

I am impressed with the simplicity and comparatively small expense required for a model test of this sort. This has been my experience in other cases where certain problems which do not involve a great deal of money are still sufficiently important so that the expenditure of the small amount required for a laboratory investigation is definitely worth while.

Of course it is necessary to have a client or owner who will cooperate to this extent. I was fortunate in my employers, the Barre Wool Combing Company and their Manager, Mr. L. M. Yacubian, in their willingness to do all that was recommended, both in the way of preliminary investigations in the field and this laboratory work on the spillway. The work was done under my general supervision and was under the direct charge of Mr. M. J. Wytrwal, Assistant Plant Maintenance Engineer at the mill. The work was also supervised by Mr. L. O. Marden, County Engineer of Worcester County who approved the original plans and who was a frequent visitor. The construction work was done by the Adams & Ruxton Construction Company of Springfield which was doing extensive work at the mill.

It is proposed to complete the work by the installation of stanchion flashboards 4 ft. high on this section of the dam. These will be operated from a bridge supported by the abutments and two piers on the dam. This work has not yet been started.

OF GENERAL INTEREST

EXCURSION OF SANITARY SECTION TO EAST- MAN GELATINE WASTE TREATMENT PLANT AND THE SOUTH ESSEX GRIT AND GREASE CHAMBERS

On Saturday, June 7, 1947, twenty-seven members and guests of the Sanitary Section visited the waste treatment plant of the Eastman Gelatine Company at Peabody, Mass., and the grit and grease chambers of the South Essex Sewerage Commission at Salem. Both

of these projects were in the construction stage.

Following the inspection of the above plants the group visited the existing South Essex pumping station and existing grit and grease chambers located near Salem Willows.



WASTE TREATMENT PLANT OF EASTMAN GELATINE CORP.

The group enjoyed a shore dinner at Salem Willows and then broke up into small parties, some of which visited the Salem Beverly water filtration plant and the others, points of historical interest in Salem.

We are indebted to Mr. Abraham C. Bolde, Assistant Sanitary Engineer of

the Massachusetts State Department of Health for the following description of the Eastman Gelatine waste treatment plant. We are also indebted to Mr. Frank L. Flood, partner of Metcalf & Eddy for the following description of the Salem Peabody Grit and Grease Chambers.

WASTE TREATMENT PLANT OF EASTMAN GELATINE CORPORATION

By A. C. BOLDE

The plant of the Eastman Gelatine Corporation is located in Peabody, Mass. All wastes from the plant are discharged into the municipal sewers which in turn discharge into the South Essex Sewerage System. Sewage in the South Essex system flows by gravity to a pumping station located near Cat Cove in Salem and from there is discharged through a deep water outfall near Great Haste Island in Salem Harbor.

Scale formation on the inside of the main sewers in Peabody and Salem and in the trunk sewer of the South Essex system is a serious problem. One factor in the scale formation is the large volume of caustic lime contained in certain of the industrial wastes discharged to the sewers.

Mr. H. W. Clark, former Chief Chemist of the Massachusetts Department of Public Health assisted by the late G. O. Adams, Assistant Chemist, and by the writer, developed a process to remove the caustic lime. This process consists of passing flue gas through the waste. The carbon dioxide present in the flue gas combines with the calcium hydroxide in the waste to form calcium carbonate which being insoluble is precipitated. The flue gas is obtained from the stacks of the industrial plants and after passing through soot removing apparatus the gas is compressed by means of rotary compressors.

The South Essex Sewerage Board has adopted a regulation prohibiting the discharge into the sewers of wastes containing more than 75 parts per million of caustic lime. Some nine carbonating plants have been established for the removal of caustic lime of which the largest is that at the Eastman Gelatine Corporation in Peabody.

The original carbonating plant at the Eastman plant consisted of a Dorr clarifier of 500,000 gallons capacity and a 300,000 gallon capacity carbonating tank. Flue gas was compressed by means of Root blowers and was diffused in the carbonating tank by means of funnel shaped orifices. The precipitated lime settled out in the last section of the carbonating tank and was removed manually. These works became inadequate and it was necessary to provide a larger and more modern treatment plant.

New Waste Treatment Plant

The new waste treatment plant consists of a 90 ft. diameter Dorr thickener of 540,000 gallon capacity for primary settling, a rectangular carbonating tank 100 by 40 by 12½ ft. deep of about 375,000 gallon capacity, and 60 ft. diameter Dorr tank of 240,000 gallons capacity for secondary settling. Flue gas is to be compressed by means of two blowers each of 1000 cubic feet per minute capacity. The gas will be diffused through 646 fun-

nel shaped orifices each $8\frac{1}{2}$ inches in diameter.

A pilot plant was operated by the Eastman Gelatine Corp. in 1945 and various analyses of the raw and treated wastes were made by the Department of Health. Analysis showed that settled waste before carbonation had a

pH of 10.6 and contained 5600 parts per million of caustic alkalinity. Carbonated effluent had a pH of 6.8 and a caustic alkalinity of 0. As a result of these tests it is expected that the new works will furnish an effluent which will be practically devoid of caustic lime.

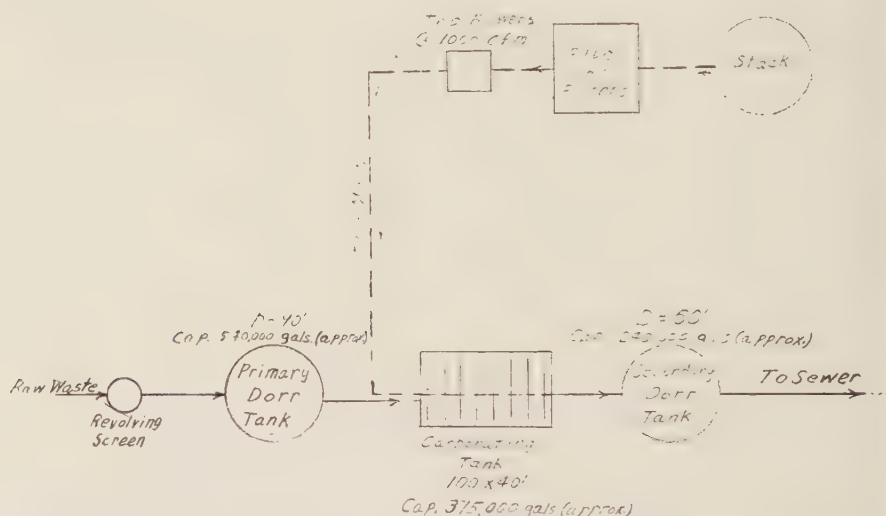


DIAGRAM SHOWING TREATMENT WORKS FOR REMOVAL OF CAUSTIC LIME AT EASTMAN GELATINE CORPORATION.

GRIT AND GREASE CHAMBER AT SALEM AND PEABODY

BY FRANK L. FLOOD

The cities of Salem and Peabody discharge their wastes into a common trunk sewer maintained and operated by the South Essex Sewerage Board. This sewer which is 52 inches in diameter at the Salem-Peabody line passes through Salem and gradually increases in size to 72 inches at the pumping station. The distance from the Peabody line to the pumping station is 14,870 ft.

In Peabody in 1943 according to the Massachusetts Department of Labor and Industry there were a total of 94 industrial establishments of which 56 processed leather in one form or other. For many years, large quantities of heavy solids and grease from these industrial plants have been discharged into the trunk sewer. Despite constant cleaning operations, it has been found impossible to keep the sewer sufficient-

ly free of deposits and incrustations to maintain adequate carrying capacity. As a result of the reduced carrying capacity, it has been found necessary to discharge quantities of wastes directly from the mills to the North River, also considerable sewage escapes from the relief over-flows along the trunk sewer.

In 1945 the Massachusetts legislature passed an act which included a provision for the construction of a grit and grease chamber near the Salem-Peabody line as a measure to reduce the deposits and incrustations in the trunk sewers. Plans and specifications for the grit and grease chamber were prepared by Metcalf & Eddy, consulting engineers during 1946. The structure was designed to provide a means of readily removing the grit and other heavy suspended solids and the grease and scum forming materials from the sewage.

Design. Peabody is served by a system of separate sewers and the area tributary to the trunk sewer is about 1300 acres and has a resident population of about 22,000. In 1945 the total sewage flow based on week-day industrial flows and on wet-weather ground-water allowances, was estimated as follows:

	Average mgd.	Maximum mgd.
Domestic sewage	1.5	3.0
Industrial wastes	5.9	13.0
Ground water	2.0	2.0
Total	9.4	18.0

Two channels are provided, affording a detention period of 22½ minutes at the average design rate of flow of 11.5 mgd. plus storage capacity for about 250 cu. yd. of grit, sludge and scum. Each channel is about 99 ft. long, 16 ft. wide, and of 14 ft. 8 in. maximum water depth.

Four drainage connections are provided to each channel near the outlet end. The upper connections may be

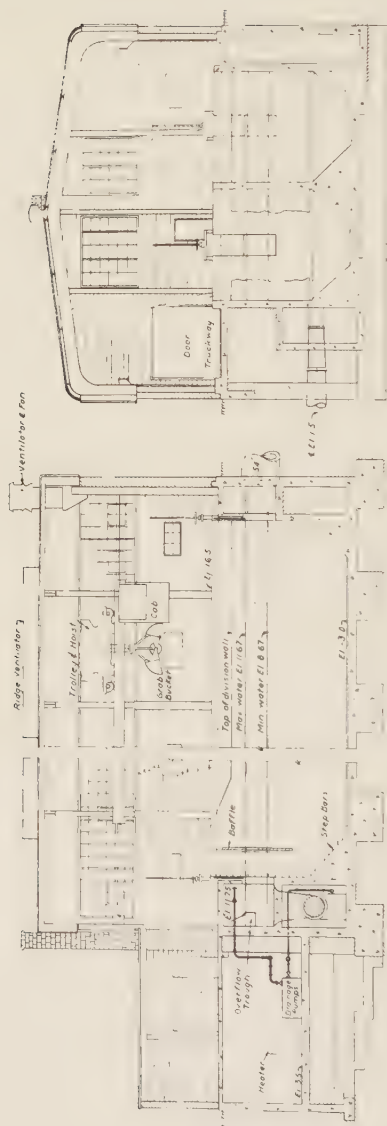
used for drawing off liquor above the sludge and the lowest connection can be used for draining the channel as desired. Two drainage pumps have been provided, each of 400 gpm. capacity. The pumps may be operated together to give a capacity of 750 gpm. or more if found desirable. One pump will remove the liquid from one channel in about 4½ hours and the two pumps operating together will accomplish the same result in about 2½ hours. The pumps are suitable for handling sewage solids and are equipped with self-priming devices. The pumps and piping have been provided with appropriate cleanouts and flushing connections.

A truckway is provided along one side of the channels. Scum and sludge will be removed from the channels by means of a ½ cu. yd. clamshell bucket operated from an overhead crane. The crane is motor-operated for travel longitudinally along the channels and for travel across the channels and over the driveway. The channels and loading platform are covered. A venturi meter is provided for measurement of the flow.

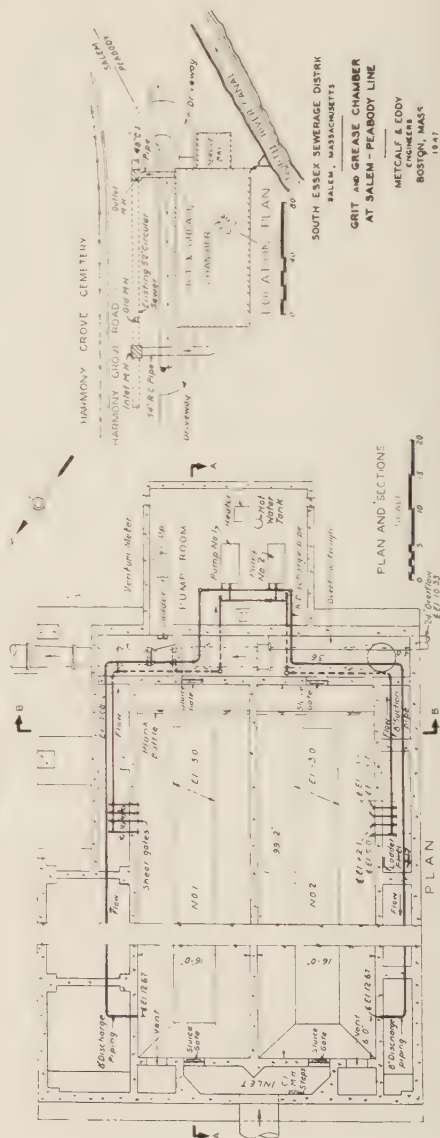
Method of Operation. Normally both channels will be in operation. Grease and other floating solids will be retained by a baffle near the outlet end of the channels. Settling solids will be collected in the bottom of the channels.

Consolidated grease and scum floating in the channels can be removed by the means proposed without taking the channel out of service and without first removing the liquid in the zone between the sludge and scum. However, for removal of grit and sludge it is proposed that one channel at a time be taken out of service and the liquid material removed by the drainage pumps and discharged to the inlet end of the other channel, which remains in service.

Watertight trucks similar to those now used in connection with transportation of materials from the present



SECTION A-A



PLAN AND SECTIONS

SOUTH ESSEX SEWERAGE DISTRICT
SALEM, MASSACHUSETTS

GRIT AND GREASE CHAMBER

WETCALF & EDDY
ENGINEERS
BOSTON, MASS
1947

Salem grit chamber will be driven into the building and the material removed by the buckets will be deposited in the trucks.

Each truck after filling will be washed down so as to be clean and unobjectionable to sight and smell as it leaves the grit chamber. Any spillage on the tops of walls, driveway and walks will be flushed back into the channels.

Disposal of Grit and Grease. The material removed by grab-bucket from the grit chambers will be transported for disposal in dumps until such time as facilities become available for its incineration.

Construction. A contract for the construction work was awarded on the basis of competitive bids to Munroe-Langstroth, Inc., North Attleboro, Massachusetts, the low bidders, at a lump sum price of \$209,163. Work was started on June 13, 1946. Completion was scheduled for February 17, 1947, but was held up by delay in delivery of cast iron pipe. Major items in the breakdown of the contractor's cost estimate are as follows:

Excavation, sheeting and dewatering, 7,000 cu. yd. at \$8.00	\$56,000
Concrete, 1,690 cu. yd. at \$30.00	50,700
Reinforcing steel, 83 tons at \$120.00	9,960
Structural steel, 72 tons at \$220.00	15,840
Brick masonry, 108 M at \$70.00	7,560
Cast-iron pipe, valves and fittings	10,000
Sewage meter, recorder and liquid level gage	5,250
Traveling crane and clam-shell bucket	12,700
Plumbing and heating	3,000
Electric	2,650
Wire fence	2,000
Drainage pumps	1,600

Limitation of the site, high ground water level and character of soil encountered in the excavation made it advantageous to use sheet steel piling and well points.

The work is under the jurisdiction of the South Essex Sewerage Board, Joseph C. Tomassello, Chairman, Elihu A. Hershenson, Treasurer and Clerk, A. Preston Chase, Frank J. McCarthy, Frank P. Morse and Albert H. Richardson, with Chester L. Nyman, District Engineer. Metcalf & Eddy is represented by William W. Lundgren, resident engineer.

FRANK A. BARBOUR

1870-1947

On May 24, 1947, the death of Frank A. Barbour, Past President, closed another career of great prominence among the membership of our Society.

Frank was born at St. John, New Brunswick, in 1870, graduated from the University of New Brunswick in 1888 and received his early training in engineering in that Province of the Dominion of Canada. Early in his career he joined the engineering staff of the Water Department of the City of Boston where he was associated with Desmond Fitzgerald and Frank McInnis, the latter a graduate of New Brunswick University.

In 1892 Mr. Barbour was employed by the City of Brockton in the design, construction and operation of sewage disposal works for that city, working under the direction of F. H. Snow, City Engineer. In 1900 Mr. Barbour and F. H. Snow opened a consulting engineering office in Boston which Mr. Barbour conducted until his death. Frank Barbour was a specialist in matters of water supply and water purification, sewerage and sewage disposal practice and appraisal of water works properties. This work was that which he most loved and, although President

of the New England Pressed Steel Company, most of his time was devoted to Sanitary Engineering Practice.

Mr. Barbour served as President of the American Water Works Association, the Boston Society of Civil Engineers and the New England Water Works Association and was an honorary member of the New England Water Works Association and the American Water Works Association. He served on the editorial advisory board of "Water Works Engineering" and served a term as Director of the Amer-

ican Society of Civil Engineers (1934-1936) during which period he served on the Committee on Publications. He was author of numerous technical articles.

Mr. Barbour was an engineer of rare ability and sound judgment and those associated with him could not but help to profit from such association. His death has resulted in a loss not only to the Boston Society of Civil Engineers but also to the engineering profession as a whole, more especially the profession of sanitary engineering.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS Boston Society of Civil Engineers

Surveying and Mapping Section

APRIL 30, 1947.—A meeting to establish this new section was held this evening in the Society rooms, 715 Tremont Temple, Boston. Thirty-seven members and friends were present.

The meeting was called to order by Mr. Frank L. Cheney, who, upon the suggestion of Mr. Edwin B. Cobb, the Society's secretary, and by common consent, acted as chairman of the group. The first business considered was that of a section name. It was voted to adopt "Surveying and Mapping Section of the Boston Society of Civil Engineers". Then Mr. Charles O. Baird, Jr., Mr. Elmer C. Houdlette and Edwin B. Cobb were appointed by the chairman to nominate officers and other members of the Executive Commit-

tee. Their choices, Frank L. Cheney for Chairman, Louis A. Chase for Vice-Chairman, Charles M. Anderson for Clerk and Hyman B. Ullian, Herman J. Shea and Ray L. Schoppe for Executive Committee, were adopted by a proper vote.

Chairman Frank L. Cheney then introduced the questions of dates and frequency of section meetings. After some discussion it was voted to meet bi-monthly beginning in September on dates to be fixed by the Executive Committee.

Mr. Charles O. Baird, Jr., Professor of Hydraulics at Northeastern University, delivered the section's first address, "A Compilation of Vertical Curve Data". Mr. Herman J. Shea led the discussion that followed.

Just before the adjournment at 8:30 P.M., Secretary Cobb, addressing the non-members present, gave a brief summary of the advantages of a membership in the Boston Society.

CHARLES M. ANDERSON, *Clerk*

APPLICATIONS FOR MEMBERSHIP

[OCTOBER 15, 1947]

The By-Laws provide that the Board of Government shall consider applications for membership with reference to the eligibility of each candidate for admission and shall determine the proper grade of membership to which he is entitled.

The Board must depend largely upon the members of the Society for the information which will enable it to arrive at a just conclusion. Every member is therefore urged to communicate promptly any facts in relation to the personal character or professional reputation and experience of the candidates which will assist the Board in its consideration. Communications relating to applicants are considered by the Board as strictly confidential.

The fact that applicants give the names of certain members as reference does not necessarily mean that such members endorse the candidate.

The Board of Government will not consider applications until the expiration of fifteen (15) days from the date given.

For Admission

ROBERT W. ANDERSON, Beverly, Mass. (b. March 20, 1921, Waterport, New York). Graduated from Mass. Institute of Technology in 1943, received B.S. degree in Building Engineering & Construction. Experience, during educational period worked 15 months at Eastman Kodak Company doing electrical and general maintenance work; February, 1943 to May, 1944, with Douglas Aircraft Company, design and analysis of aircraft structures; June, 1944 to June, 1946 (Lt. j.g.) U. S. Navy Department of Public Works at Naval Supply Center, Guam, supervision of construction and maintenance of buildings and services; June, 1946 to September, 1946, with Cleverdon,

Varney & Pike, Boston, structural design of buildings utilizing light steel frame construction; September, 1946 to date, Ganteaume & McMullen, Boston, structural designer doing reinforced concrete, structural steel and timber design. Refers to *A. G. H. Dietz, D. Peabody, Jr., W. C. Voss, H. I. Wyner.*

WALTER C. ANDERSON, Brockton, Mass. (b. June 18, 1899, Brockton, Mass.) Completed grammar and high schools at Brockton. Appointed to assistant chemist Brockton Sewage Treatment Plant, June, 1917; attended chemical engineering course at Northeastern University for two years (evenings). In 1944, successfully passed Civil Service requirements for position of chemist, Brockton Sewage Treatment Plant and appointed to position. Refers to *E. B. Cobb, G. G. Bogren, W. E. Merrill, R. M. Soule.*

PETER R. BAGARELLA, Watertown, Mass. (b. November 7, 1912, Boston, Mass.) Graduated from Boston High School of Commerce, June, 1929. Lincoln Tech. Institute, June, 1935, receiving diploma in Civil Engineering. Tufts College Engineering School (evenings) January, 1941 to May, 1941 also October, 1941 to May, 1942, receiving certificates in Concrete Construction and Structural Design and Detailing. Experience, with Corps of Engineers, New England Division, October, 1936, to present (except from October, 1947, to January, 1947, employed by Metcalf & Eddy, as Mechanical and Structural Draftsman) Chief Draftsman and assistant to the Project Engineer making studies. Designs and Layouts of Airports and Flood Control Projects. Refers to *A. J. Burdoin, E. F. Childs, R. J. Rice, H. I. Wyner.*

ARNOLD C. BLAKE, Lowell, Mass. (b. July 10, 1908, Hill, New Hampshire). Attended Tilton Prep. School

Tilton, N. H., in 1925. Graduated from the University of New Hampshire in 1930, received B.S. degree in Mechanical Engineering. Experience, 1930-1933, Mechanical Engineer, Western Union Telegraph Company, N. Y.; 1933-1936, N. H. Public Service Commission, assistant hydraulic engineer; 1936-1938, Nepsco Services Inc., field engineer, Dams & Power Stations; 1938-1940, Pittsburg Dam, assistant resident engineer, N. H. Water Resources Board; 1940 to date, Field Engineer with Chas. T. Main, Inc., Boston. At present office engineer. Refers to *F. M. Gunby, C. T. Main, 2nd, L. G. Ropes, A. T. Safford.*

DEAN F. COBURN, South Weymouth, Mass. (b. December 12, 1914, Newport, Vermont). Graduated from University of Vermont in June, 1936. Experience, August, 1936 to December, 1938, inspector and topographic draftsman with Vermont Highway Department; December, 1938 to August, 1941, engineering aide and junior engineer in hydraulics section of U. S. Engineer Office, Providence, R. I., working on flood control projects; August, 1941 to December, 1945, U. S. Army—Infantry; December, 1945 to April, 1947, assistant engineer with Hydrology Section, U. S. Engineer Office, Providence, R. I., and Boston, Mass., working on flood control of the Connecticut River Basin; April, 1947, to date, assistant engineer with Metcalf & Eddy, Boston, Mass. Refers to *E. F. Childs, S. Haseltine, A. L. Shaw, L. W. Ryder.*

ROBERT E. CRAWFORD, Malden, Mass. (b. August 26, 1905, Chelsea, Mass.) Graduated from Massachusetts Institute of Technology in 1928, B.S. degree in Civil Engineering. Experience, 1928-1929, structural steel detailer, American Bridge Company Trenton, New Jersey; 1929-1932, designer, Department of Structures, New York Central Railroad, N. Y.; 1933-1934, 1942-1944, assistant engineer, Fay, Spofford &

Thorndike, Boston, Mass.; 1934-1942, assistant engineer, bridges, Southern Railway, Knoxville, Tenn.; 1944-1945, engineer, Haller Engineering Associates, Cambridge, Mass.; 1945-1946, engineer, Charles W. Riva Company, General Contractors, Boston; 1946, engineer, Gil Wyner Company, General Contractors, Boston; 1946 to date, structural engineer in charge of structural department, Fay, Spofford & Thorndike, Boston, Mass. Associate Member of ASCE; Member Northeastern Section ASCE; Professional Engineers license in New York State and Massachusetts. Refers to *C. A. Farwell, W. L. Hyland, M. H. Mellish, C. M. Spofford, J. B. Wilbur.*

WILLIAM E. DOBBINS, Woburn, Mass. (b. March 1, 1913, Woburn, Mass.) Graduated from Woburn High School in 1930. Registered at Mass. Institute of Technology in September, 1930, and received the degrees of S.B. in Sanitary Engineering in 1934, S.M. in Sanitary Engineering in 1935 and Sc.D. in Sanitary Engineering in 1941. Experience, September, 1935, to June, 1936, and from September, 1939, to June, 1940, employed as Research Assistant in the Department of Civil and Sanitary Engineering at the Mass. Institute of Technology; September, 1936, to June, 1939, employed as Instructor in Hydraulics and Sanitary Engineering at Robert College, Istanbul, Turkey; August, 1940 to February, 1941, employed as an engineer with the late S. M. Ellsworth, Cons. Engineer, Boston; September, 1941, to January, 1942, was engaged as an engineer on the design of water supply and waste disposal systems with Stone & Webster Engineering Corp., Boston; February, 1942, to August, 1943, with Foley Brothers Inc., and Spencer, White and Prentis, Inc., as field engineer on dock construction and water supply in the Persian Gulf area; September, 1943, enlisted in the U. S. Navy and was

separated at Lt. (j.g.), CEC, USNR in June, 1946. From June, 1946 to date has been an engineer with Fay, Spofford & Thorndike, Engineers, Boston. Refers to *F. M. Cahaly, T. R. Camp, R. W. Horne, C. R. Spielvogel*.

CHARLES V. DOLAN, Wakefield, Mass. (b. February 22, 1908, Everett, Mass.) Graduated (Engineering (Civil) Administration) M.I.T. 1931. Experience, 1931-1935, Minor Surveying (house lots, etc.), built 6 room house, civil engineer CWA Project, draftsman ERA Project, consulting engineer on Traffic and Parking problems for Chamber of Commerce Merchants groups; 1935-1940, experience other than engineering, i.e., Department Store Executive; 1940-1942, 1944-1946, U. S. War Department (Zone 1, constructing QM, North Atlantic Division Engineer, 1st Service Command Engineer) drafting, field inspections, progress reports, completion reports, cost accounting; 1942-1943, the Prebilt Company, General Contractor, Housing Projects, cost engineering, estimating, scheduling, purchasing, progress reports; 1943-1944, Holtzer, Cabot Electric Company, Mechanical Engineer, time and motion study, fixture design; 1946 to present, Fay, Spofford & Thorndike, Architect Engineer, Alaska Project, Assistant Engineer. Refers to *C. A. Farwell, W. L. Hyland, M. H. Mellish, C. M. Spofford*.

FOSTER D. DUCHARME, W. Somerville, Mass. (b. June 25, 1916, Arlington, Mass.) Attended high school in Montreal, Science Course, 1934; Boston Navy Yard shipbuilding course, 1940. Experience as helper shipfitter and shipfitter at Boston Navy Yard from September, 1939, to March, 1941; Apprentice draftsman to Principal Engineering Draftsman at Boston Navy Yard on ship piping March, 1941 to November, 1945; Designer at E. I. DuPont de Nemours, Wilmington, Delaware, on process piping and equip-

ment layout, November, 1945, to January, 1947; Designer at Stone & Webster on mechanical equipment February, 1947 to June 1947; Designer at Fay, Spofford & Thorndike, sanitary division, June, 1947 to date. Refers to *H. L. Crocker, F. L. Heaney, W. L. Hyland, M. H. Mellish*.

JEAN M. DUCHARME, East Walpole, Mass. (b. November 16, 1910, Boston, Mass.) Graduated from Rensselaer Polytechnic Institute, Troy, New York, in 1931 with the degree of Civil Engineer. Experience, 1930-1931, surveying for the Delaware & Hudson R.R. Corporation; October, 1932 to March, 1934, employed by the Hollinger Consolidated Gold Mines, Ltd., Timmins, Ontario, as assistant Mine Surveyor and later a miner; April, 1934, went to Honduras as Resident Engineer, designing and supervising construction of irrigation, drainage and flood control systems for the United Fruit Company and later for the Standard Fruit Company. Returned to the United States in 1936 and employed by J. R. Worcester & Company, starting out as surveyor and draftsman, by the fall of 1938 was designer in charge of a branch office handling bridge surveys and designs after the hurricane and floods of that year. During this same period, also gained considerable experience in the design of rigid frame structures while working for Jackson & Moreland. May, 1939, became associated with the Crandall Dry Dock Engineers, Inc., as designer, Resident Engineer, Construction Superintendent and at present as Engineer of Design in charge of the design of all projects (railway and floating dry docks) in the United States, Canada, Iceland, Europe, Latin America, etc. Refers to *J. S. Crandall, J. W. Howard, O. G. Julian, R. H. Lindgren*.

HARRY R. FELDMAN, Boston, Mass. (b. July 27, 1910, Framingham, Mass.) Graduated from Framingham High

School in June, 1928. Received B.C.E. degree from Northeastern University in June, 1932. Experience, 1932-1933, worked as a maintenance engineer for the Hodgman Rubber Company, Framingham; 1933-1938, employed by Town of Framingham Engineering Department — Municipal Engineering; 1938-1940, Metropolitan District Water Supply Commission, general construction inspector on project between Southboro and Weston; 1940-1943, employed by Chas. T. Main, Inc., Railroad Engineer, Assistant to Water and Sewer Engineer; 1943-1946, Civil Engineer Corps, U. S. Naval Reserve; 1946 to date in business for myself practicing Civil Engineering. Refers to *A. W. Benoit, F. M. Gunby, B. O. McCoy, C. T. Main, 2nd, T. H. Safford.*

WILLIAM E. GOODWIN, Boston, Mass. (b. August 14, 1899, Cambridge, Mass.) Attended Rindge Tech. 1914-1917; 1917-1919, U. S. Army; 1919-1920, Rindge Tech.; 2 years night refresher courses at Tufts, 1940-1941; Experience, 1921-1938 with E. Goodwin & Company; 1938-1940, Mass. Dept. Public Works, Civil Engineer, drafting; 1940-1942, Fay, Spofford & Thorndike, drafting; 1942-1944, E. B. Badger & Sons, high pressure piping draftsman; 1944-1947, Raytheon Mfg. Company, mechanical layout and design engineer; 1947, Fay, Spofford & Thorndike, designer. Refers to *L. M. Gentleman, F. L. Heaney, W. L. Hyland, M. H. Mellish.*

PHILLIP R. JACKSON, Allston, Mass. (b. December 3, 1921, Roxbury, Mass.) Graduated from Dartmouth College in 1942 and received a degree of Civil Engineer from the Thayer School of Engineering in June, 1943. Entered the Naval Service and trained for Aeronautical Engineering duties. Upon discharge from service in July, 1946, entered the employment of Fay, Spofford & Thorndike, duties involve drafting,

estimating, detailing and design in structures and piping. Refers to *F. M. Cahaly, G. C. Douglas, K. R. Garland, C. R. Spielvogel.*

WALTER I. LEWIS, JR., Wakefield, Mass. (b. July 3, 1918, East Liverpool, Ohio). Received B.S. degree from Rutgers University College of Agriculture in June, 1941, with a specialization in Forestry and a minor in economics. Experience, 1941, in the purchasing department of The Anaconda Wire and Cable Company of Hastings, N. Y.; September, 1942, entered the Army of the United States and assigned to the Air Forces and trained and served as an Airport Control Tower Operator until appointment into the Army Specialized Training Program. Under this program received four thirteen weeks terms of Civil Engineering at Pennsylvania State College and two terms (six months) of Sanitary Engineering at Harvard University. On completion of ASTP courses was accepted for Officer Candidate School and upon graduation from there in April, 1945, was commissioned as a Second Lieutenant in the Sanitary Corps and assigned to the Philippine Islands until my separation in April, 1946. September, 1946, accepted employment with the Commonwealth of Massachusetts, Department of Public Health, Division of Sanitary Engineering, as Senior Sanitary Engineering Aide; September, 1947, with Metcalf & Eddy, Boston, Mass. Refers to *A. J. Burdoin, F. H. Kingsbury, W. E. Merrill, R. M. Soule, E. Wright.*

ALFRED A. LOCKERBIE, Marblehead, Mass. (b. November 9, 1907, Marblehead, Mass.) Graduated from Northeastern University in 1931, with B.S. degree in Civil Engineering. Experience, May, 1928 to May, 1929, Newell B. Snow, Engineer, Buzzards Bay, Mass., rodman and transitman on highway, transmission lines, private surveys.

Some computing and drafting; May, 1929 to Dec., 1930. Boston & Maine Railroad, Maintenance of Way, Dover, N. H., rodman and transitman on track layout, bridge surveys, assisting resident engineer on general construction, Worcester, Mass. Yard expansion program. Computing and some drafting; F. A. Barbour, Engineer, Boston, Mass., as transitman, topographic survey, sewer interceptor location, Town of Marblehead. Inspector Sewer Installation, Town of Marblehead (both temporary—two to three weeks); Foss-Goodwin Inc., Marblehead, Mass., intermittent general surveying, drafting and computing; Dept. of Commerce, U. S. Coast & Geodetic Survey, Provincetown, Mass., May, 1933 to Nov., 1933, Coxswain Hydrographic Launch. Assisting on surveys; C.W.A. Project #3351, Leggs Hill Road, Marblehead and Salem, Mass., two weeks Dec., 1933, inspection and assistance on survey; Dept. of Commerce, U. S. Coast & Geodetic Survey, Charleston, S. C., Dec. 1933 to Aug. 1935, drafting on Hydrographic Surveys. Computing triangulation. Some observing. Raised to rating of Surveyor, Oct. 1931; Mass. Geodetic Survey, Department of Public Works, Boston, Mass., Dec. 1935, adjusting triangulation by method of least squares. Computing plane coordinates on Lambert conformal Conic Projection; U. S. Engineer, Buzzards Bay, Mass., Apr. 1936 to Sept. 1936, sub-inspector on excavation and revetment Cape Cod Canal. Assisting on miscellaneous surveys. Some drafting. Pay quantity computations: Mass. Geodetic Survey, Boston, Mass., Nov. 1936, computation of triangulation and adjustment. Magnetic Declination Computations. Compilation of a table of intersections of Latitudes and Longitudes for converting to Lambert Conformal Conic Projection; Town of Marblehead, Engineering Department, April 23, 1937 to date, Assistant Town Engineer from August, 1938. Duties

consist of preparing detailed estimates for projects; conducting surveys; drawing plans and any designs required; computation of all surveys, road layouts, etc., writing contracts and specifications in detail. Military Leave of Absence, March 23, 1944 to December 17, 1945, multiplex operator with 2765th Engineers Base Photo-mapping company, Graduate of Phototopography School, Ft. Belvoir as Aerial Phototopographer. Refers to *C. O. Baird, E. S. Averell, H. M. Wilkins, E. C. Houdlette.*

HAROLD MOHN, Norwood, Mass. (b. September 25, 1885, Oslo, Norway). Graduated from high school, Oslo, Norway, 1903, and from the Norwegian National Forestry School, 1908. Member of Norwegian-Swedish Boundary Commission, 1909. Extension Forester, 1909-1912. Experience, 1912-1916, employed on surveys, rod man, transit man, chief of party. Surveys comprised forest, farm land, and waterways (power development), Norway; 1917-1922, reconnaissance surveys and forestry work in Newfoundland. Newfoundland Shipbuilding Co. Ltd., Anglo-Newfoundland Development Co. Ltd.; 1922-1927, fire insurance surveyor and draftsman, Boston, Plan Department of the Associated Factory Mutual Fire Insurance Companies; 1928-1930, Senior draftsman, engineering department, Massachusetts Land Court; 1930-1931, self employed in business; since Pearl Harbor, up to present time employed as topographical draftsman on U. S. Government projects, Lockwood-Green, Engineers., J. R. Worcester & Company, Fay, Spofford & Thorndike. Refers to *C. A. Farwell, W. L. Hyland, M. H. Mellish, T. Worcester.*

EDWARD W. MOORE, Cambridge, Mass. (b. December 5, 1906, Middleboro, Mass.) Received degrees of A.B. (1929) and A.M. (1931) from Harvard University in the field of Chemistry. Experience, employed in 1929 by the

Harvard Graduate School of Engineering, and have since that time been engaged in teaching and research in Sanitary Engineering at that institution, except for the period 1943-1946 during which served overseas in Sanitary Corps U. S. Army, in grade of Lt. Col., as Associate Professor of Sanitary Chemistry. Refers to *T. R. Camp, F. Kingsbury, G. M. Fair, J. E. McKee, H. A. Thomas, Jr.*

GARDINER E. SMITH, Randolph, Mass. (b. November 5, 1908, Elmira, New York). 1921-1925, attended Elmira Free Academy. 1925-1929, entered the Apprentice School of Structural Engineering at the American Bridge Company, Elmira, New York. Experience, 1929-1932, 1934-1936, structural draftsman for the American Bridge Company; March, 1936, accepted a position with the Bethlehem Steel Company, at their Pottstown office and remained there through October, 1936, at which time was transferred to the Bethlehem Company's Shipbuilding Plant at Quincy, Mass., in the structural department of the drafting rooms from 1936 to March, 1947. From March, 1947 to date with Fay, Spofford & Thorndike, in the drafting department. Refers to *F. M. Cahaly, G. C. Douglas, K. R. Garland, F. L. Lincoln.*

WILLIAM P. SOMERS, Lynn, Mass. (b. August 29, 1915, Revere, Mass.) Graduated from Tufts College in 1938 with B.D. degree in Civil Engineering. Experience, October 1938, began work with the Water Resources Branch, Boston District office, of the United States Geological Survey, obtaining field data for determination of flood flows following the 1938 hurricane. December, 1938, appointed Jr. Engineer on a temporary basis, and in August, 1939 on a permanent basis. Following flood flow computations in the office early in 1939, a field area was assigned where it was my responsibility to obtain stream flow data. This involved making current-meter meas-

urements of discharge, operation and maintenance of continuous recording gaging stations, and subsequent analysis and processing of field data. During 1940 engaged in a study involving backwater effects of river channel obstructions and the possible improvements effected by revisions of channels, bridge openings, and floodways. Following that study, returned to the collection, analysis, and processing of stream flow data. At present am occupied in the preparation of a report undertaken by this office on hydrology of Massachusetts. An extensive compilation of hydrologic records will be followed by an exhaustive analysis of the data. Advance in grade from Junior Engineer was made in August, 1942, and a second advance was made in January, 1947. Refers to *H. P. Burden, C. E. Knox, H. B. Kinnison, F. N. Weaver, G. K. Wood.*

ARIEL A. THOMAS, Arlington, Mass. (b. June 20, 1914, Woonsocket, R. I.) S.B. degree in Civil and Sanitary Engineering from Massachusetts Institute of Technology in 1936; M.S. degree in Sanitary Engineering from University of Illinois in 1938. Experience, July, 1938 to June, 1941, district sanitary engineer, Division of Sanitary Engineering, Illinois Department of Public Health; June, 1941 to March, 1943, Senior Sanitary Engineer in the Sewerage Section of Division of Sanitary Engineering of Illinois Department of Public Health; March, 1943, to February, 1946, 1st Lt., Capt. and Major in Sanitary Corps of U. S. Army. Some of my assignments were Sanitary Inspector at Camp Edwards and at Presidio of San Francisco, and Division Medical Officer of the 104th Infantry Division; February, 1946 to July, 1947, Instructor of Sanitary Engineering at Mass. Institute of Technology; July, 1947 to date, Assistant Professor of Sanitary Engineering at Massachusetts Institute of Technology. Refers to *T. R. Camp, H. G. Dresser, W. E. Stanley, J. B. Wilbur.*

HAROLD O. W. WASHBURN, Somerville, Mass. (b. September 1, 1915, Swampscott, Mass.) Experience, October, 1941, to October, 1943, at Boston Navy Yard, as instructor in charge of blueprint classes—Marine Piping and Machinery installation; 1943 to 1945, Marine Engineer (Lt.) with the U. S. Maritime Service; three months as draftsman with Stone & Webster, Boston, Mass.; December, 1945, to March, 1946, returned to sea with U. S. Maritime Service as First Assistant Engineer until December, 1946; January, 1947, to May, 1947, with Public Works Division, Navy Department, Great Lakes, Illinois, as Mechanical Designer; June, 1947 to date with Fay, Spofford & Thorndike, Boston, Mass., as draftsman on Alaskan Project. Refers to *C. A. Farwell, F. L. Heaney, W. L. Hyland, M. H. Mellish.*

NATHANIEL N. WENTWORTH, JR., Canton, Mass. (b. February 11, 1917). Received B.S. degree in Civil Engineering from Rhode Island State College in June, 1939. Experience, employed by Fay, Spofford & Thorndike, as junior draftsman and remained in their employ until February, 1942, when ordered to active duty with the Army of the United States. Entered the Army in March, 1942, as Second Lieut. of Infantry, was promoted to First Lieut. and Captain. Honorably discharged in August, 1945. Returned to the employ of Fay, Spofford & Thorndike as draftsman and have remained to present date. Refers to *F. M. Cahaly, G. C. Douglas, K. R. Garland, F. L. Lincoln.*

HENRY R. WHEELER, JR., Baldwinville, Mass. (b. July 21, 1921, Winchendon, Mass.) Graduated from Norwich University in 1943 with B.S. degree in Civil Engineering. Immediately following graduation in March 1943 to August 1947, served with Corps of Engineers, U. S. Army in United States, England, Wales, France, and Belgium. Served as platoon and company commander in 342 Engr. Gen. Serv. Regt.

on Communication Zone construction projects in the European Theatre of Operations for two years. Supervised construction work on such projects as: highways, railroads, airports, tented camps, quarry operations, and camp maintenance work. Employed such equipment as tractor bulldozer, motorized air compressors with pneumatic tools, power saws, cranes, power shovels, concrete pavers, etc. Currently employed by Metcalf & Eddy, engineers, as an inspector on sewer projects, of which the company is supervising the construction. Junior, American Society of Civil Engineers, 1943. Refers to *E. B. Cobb, A. J. Burdoin, F. L. Flood, R. J. Rice.*

FORREST SHERMAN WHITE, Holliston, Mass. (b. September 21, 1903, Chelmsford, Mass.) Graduated from Mass. Institute of Technology in 1927 with degree of S.B. in Architectural Engineering. Experience, June, 1927, to September, 1930, and April, 1933 to February, 1935, with Jackson & Moreland as draftsman and structural designer, appraisal engineer, stress computer, and inspector; October, 1930 to January, 1931, with Gibbs & Hill, New York City, as designing draftsman; February, 1935, to February, 1936, Cambridge Administrative Office of E.R.A. as technical engineer; February, 1936 to October, 1946, U. S. Engineer Corps at the following locations: (a) February, 1936 to July, 1936, structural draftsman at Eastport, Maine; (b) July, 1936, to September, 1939, structural draftsman and structural engineer at Boston, Mass.; (c) September, 1939 to March, 1942, structural engineer, at Binghamton, New York; (d) March, 1942 to October 1946, structural engineer, assistant in charge of operations engineering section, in charge of compilation of information for detailed confidential reports for Washington Army Chiefs, assistant in charge of an engineering section designing and preparing contract documents for flood control

projects and last as engineer in charge of the same engineering section, at Syracuse, New York; November, 1946 to date, with Metcalf & Eddy, Boston, as designing structural engineer. Professional registered engineer in the Commonwealth of Massachusetts. Refers to *E. B. Cobb, E. H. Cameron, R. J. Rice, C. A. Richardson.*

DAVID YONA, Cambridge, Mass. (b. March 10, 1901, Ivrea, Italy). Graduated with honors as a Doctor in Civil Engineering and Architecture by the Institute of Technology of Turin, 1925. Experience, 1926-1935, designing engineer and architect and project director by the Department of Public Works of the City of Turin; 1935-1940, free lance professional engineer in Turin, Italy; 1941-1944, designing draftsman with Stone & Webster Eng. Co., Boston; 1944-1945, structural designer with E. B. Badger & Sons, Boston; 1945-1946, structural designer with Stone & Webster Eng. Co., Boston; 1946 to date, structural designer with Cram & Ferguson, Architects and Engineers, Boston. Registered professional engineer, Commonwealth of Massachusetts. Refers to *H. P. Duffill, M. J. Lorente.*

Transfer from Grade of Student

ROBERT J. KILEY, North Quincy, Mass. (b. May 20, 1918, Boston, Mass.) Graduated from Northeastern University in 1941, received B.S. degree in Civil Engineering. Experience, 1942, U. S. Naval Academy, Naval Architect; 2 years with Aspinwall & Lincoln; 1 year with Whitman & Howard, 3 months Town of Belmont, 1 month New England Foundation, 6 months Sawyer Construction; 4½ years U. S. Navy, Ens. to Lt. Comdr. ship repair; October to November, 1945, Morton C. Tuttle Company, assistant layout engineer, Erie, Pa.; November, 1945 to May, 1946, Sawyer Construction Company, General Hospital, Boston; May, 1946 to present Colonial Beacon Oil Company, Boston, field inspector, construction and maintenance department.

Refers to *C. O. Baird, L. Chase, E. A. Gramstorff, W. M. Raphael.*

ADDITIONS

Members

- Henry E. Bilodeau, 797 Bay Street, Taunton, Mass.
 Edmund H. Brown, 56 West Central Street, Natick, Mass.
 Nathaniel Clapp, Whiting Road, Dover, Mass.
 William R. Cuff, 49 Shephard Road, South Braintree, Mass.
 Charles E. Cyr, 303 Market Street, Lawrence, Mass.
 Stephen Haseltine, 27 Harrison Street, Melrose, 76, Mass.
 William B. Hilton, 27 Great Woods Road, Lynn, Mass.
 S. Albert Kaufmann, 212 Pleasant Street, Malden 48, Mass.
 Donald F. Libby, 120 Linden Street, Everett, Mass.
 Richard W. Logan, 50 Brooks Street, W. Medford 55, Mass.
 George R. Rich, Chas. T. Main, Inc., 80 Federal Street, Boston 10, Mass.
 Ray L. Schoppe, U. S. Geod. Coast Survey, 10th Fl. Customhouse, Boston 9, Mass.
 Edward N. Smith, 27 Worcester Square, Boston 18, Mass.
 Stephen N. Smith, 1988 Commonwealth Avenue, Boston 35, Mass.
 Henry E. Weiss, 694 Washington Street, Boston, Mass.

Juniors

- John M. Campbell, 23 High Street, Marlboro, Mass.
 Eugene D. Mellish, 81 Garfield Street, Cambridge 38, Mass.
 Alfred J. Pacelli, 96 Atlantic Street, No. Quincy, Mass.

DEATHS

- FRANK A. BARBOUR, May 27, 1947
 ADIN M. CUSTANCE, June 22, 1946
 ROBERT R. EVANS, Sept. 13, 1947
 ARTHUR A. FORBES, Jan. 7, 1946
 DANA M. PRATT, Aug. 19, 1947
 CHESTER W. SMITH, July 22, 1947

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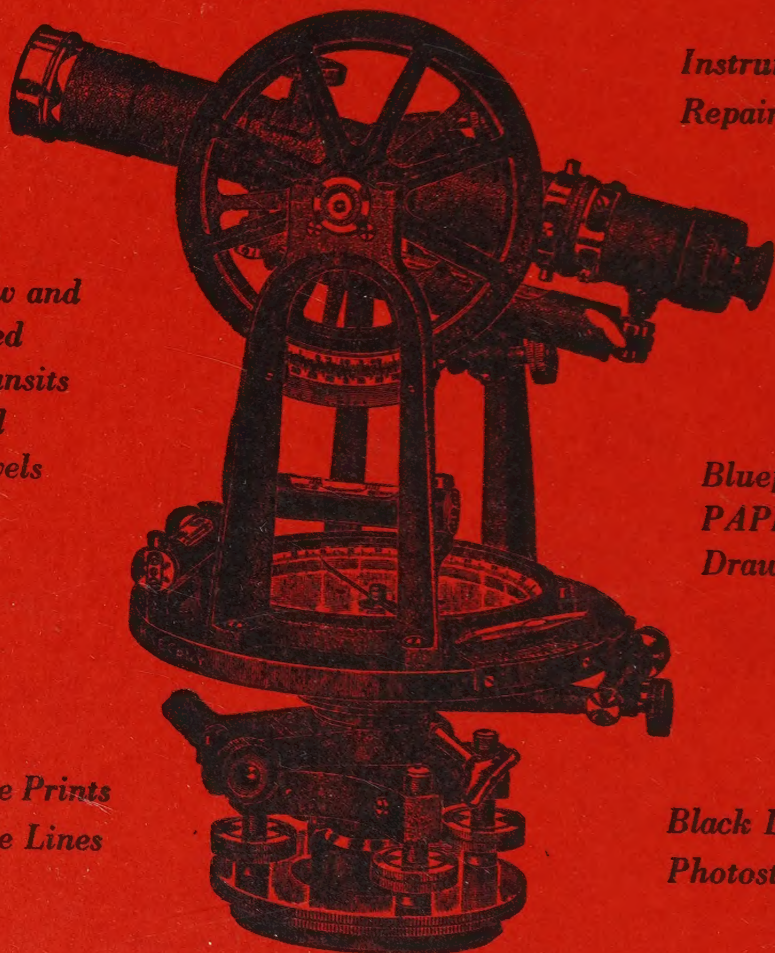
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